Design of the Surface Infrastructure Components Doris North Project, Hope Bay, Nunavut, Canada

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1 Introduction

This report presents the geotechnical design of select surface infrastructure components for the Doris North Project owned by Miramar Hope Bay Ltd. (MHBL). This report has been prepared as part of the Water Licence Application to the Nunavut Water Board (NWB), and has been updated from the October 2006 submission to include review comments received by the NWB in December 2006. The following documents form part of the designs presented in this report, and should therefore be read in conjunction with this document:

- SRK Consulting (2007a). Design of the Tailings Containment Area, Doris North Project, Hope Bay, Nunavut, Canada. Report submitted to MHBL, March.

- SRK Consulting (2007b). Technical Specifications for Tailings Containment Area and Surface Infrastructure Components, Doris North Project, Hope Bay, Nunavut, Canada. Report submitted to MHBL, March.

The drawings referenced in this report form part of a set of engineering drawings completed for the Tailings Containment Area (TCA) and surface infrastructure designs for the Doris North Project, and are bound as a separate volume:

- SRK Consulting (2007c). Engineering Drawings for Tailings Containment Area and Surface Infrastructure Components, Doris North Project, Hope Bay, Nunavut, Canada. Drawings submitted to MHBL, March.

This report includes information about the design, operation, monitoring, maintenance and construction of the following permanent surface infrastructure components, as illustrated in Dwg. G-02:

- Jetty.
- Roads, including road turnouts, culverts and caribou crossings.
- Pads for laydown areas, mill, camp, explosives storage facility, pump house, and temporary waste rock pile.
- Airstrip and apron.
- Fuel transfer station and fuel tank farm.
- Float plane dock.
- Bridge.
- Surface water management structures such as the sedimentation pond.
- Pollution control management structures such as the pollution control pond.
- Landfill and landfarm.

SRK has completed geotechnical designs of the facilities only. Ancillary facilities such as pumps, piping or any services are not part of the SRK Consulting (Canada) Inc. (SRK) scope of work. Furthermore, any structures to be placed on the pads covered by this design report are outside the SRK scope of work. Where these structures are mentioned they are for information only, such that the design concepts can be clearly understood.

2 Project Area Description

2.1 Location and Access

The Doris North Project is located approximately 400 km east of Kugluktuk and 160 km southwest of Cambridge Bay in the West Kitikmeot Region of the Territory of Nunavut. The project location is shown on Dwg. G-01.

Access to the site is by air (float planes in the summer, and an ice airstrip in the winter); with an annual barge sealift re-supply in Roberts Bay during the open water season.

2.2 Regional Geology

The Doris North Project is in the faulted Bathurst Block, forming the northeast portion of the Slave Structural Province, a geological sub-province of the Canadian Shield. The region is underlain by the late Archean Hope Bay Greenstone belt, which is seven to 20 km wide and over 80 km long in a north-south direction. The belt is mainly comprised of mafic metavolcanic (mainly meta-basalts) and meta-sedimentary rocks that are bound by Archean granite intrusives and gneisses. The greenstone package has been deformed during multiple events, and is transected by major north-south trending shear zones that appear to exert a significant control on the occurrence of mineralization, particularly where major flexures are apparent and coincident with antiforms.

2.3 Seismicity

A site specific seismic hazard assessment was done by the Geological Survey of Canada, according to the procedures documented in Adams and Halchuck (2003). Peak ground accelerations and velocities for various annual probabilities of exceedence were determined and are listed in Table 2.1.

Table 2.1: Probabilistic seismic ground motion analysis at the Doris North Project site

Annual Probability of Exceedence	Return Period (Years)	Peak Ground Acceleration (g)	Peak Ground Velocity (cm/sec)
0.01	100	0.014	0.033
0.005	200	0.018	0.039
0.0021	475	0.023	0.049
0.0010	1,000	0.028	0.060
0.0004*	2,475	0.059	-

*The 1:2,475 return period data is not site specific to the Doris North Project area, but are for Kugluktuk.

The Doris North Project falls within the “stable” zone of Canada. This region has too few earthquakes to define reliable seismic source zones. However, international experience suggests that large earthquakes can occur anywhere in Canada, although the probability is very low.

Within this “stable” zone, the project area falls in acceleration zone 1 ($Z_a = 1$) and experiences zonal accelerations of 0.05 g. The velocity zone in which the area falls is zone 0 ($Z_v = 0$) which corresponds to zonal velocities of 0.05 m/s. These zonal classifications are the lowest zones classified on the seismic hazard maps of Canada (Adams and Halchuck 2003).

2.4 Climate

Baseline climate data for the project were collected at the Boston and Windy camps during exploration (August 1993 thru 2003, with some interruptions). This site specific data combined with data from three longer-term regional weather stations operated by Environment Canada (Lupin, Cambridge Bay, and Kugluktuk) were used to develop annual climate profiles for the Doris North site.

The Doris North site has a low arctic ecoclimate with a mean annual temperature of -12.1^{o}C with winter (October to May) and summer (June to September) mean daily temperature ranges of -50^{o}C to +11^{o}C and -14^{o}C to +30^{o}C, respectively; and mean precipitation ranges from 94 mm to 207 mm, with only about 40% falling as rain. Annual lake evaporation (typically occurring between June and September) is about 220 mm.

Wind speed data reported for the Boston area (Rescan 2001) indicates predominant wind directions ranging from northwest to northeast, with wind speed in the order of 5 to 7.5 m/s. Calm conditions (wind speed below 1 m/s) occur about 6 to 9% of the time.

2.5 Permafrost

The Doris North site is underlain by continuous permafrost that has been estimated to extend to depths in the order of 550 m (SRK 2005a). This permafrost depth is based on a 200 m deep drill hole (SRK-50, see Dwg. G-04) where the mean surface temperature is about -6.3 °C and the geothermal gradient is 11.4 °C km^{-1}. The geothermal gradient in the upper 100 m appears to be isothermal or slightly negative. For comparison, the deep ground temperature profile measured at the Boston Camp, 60 km south of the site, also suggested a similar permafrost depth, about 560 m (EBA 1996; Golder 2001). The mean annual surface temperature is however colder at –10 °C and the geothermal gradient is higher at 18 °C km^{-1}. The difference in the ground temperature profiles at these two sites can be attributed to different surface conditions and the thermal conductivity of the ground at depth. The geothermal gradient measured at the Doris North site is probably representative of the conditions at Tail Lake.

Temperature data collected around Tail Lake indicates that the active layer in the marine clay/silt soils appears to be about 0.5 m, while the sand deposit has an active zone no greater than 2 m. The depth of zero annual amplitude varies between 11 and 17 m. The ground temperatures at the depth of zero annual amplitude are generally in the range of -9 to -7 °C.

2.6 Hydrology

The Doris North Project is located primarily in the Doris Lake outflow drainage basin. Tail Lake basin, part of the Doris basin, is the Projects TCA. Peak flows typically occur in June during snowmelt. A second smaller peak may occur from rainfall in late August or early September. The streams in the study area are usually frozen with negligible flow from November until May. The mean flow from June to October for Tail, Doris and Little Roberts Lake outflows are about 0.03, 0.85, and 1.73 m^3/s, respectively (AMEC 2003).

2.7 Hydrogeology

The permafrost underlying the Project site is generally impervious to groundwater movements. Groundwater movement will only occur in the shallow active layer (0.5 m to 2.0 m) during its seasonal thaw period. There is no hydraulic connection between the taliks beneath Tail and Doris Lakes, as has been demonstrated though a series of deep drill holes (SRK 2005b).

3 Design Criteria and Assumptions

3.1 Design Basis

The Doris North Project will have an operating life of two years, followed by an active decommissioning phase of about two years, for a total design life for most of the surface infrastructure of about four years. This Project is the first phase of development for MHBL in the Hope Bay Belt, and although MHBL is actively pursuing towards better definition of any future development phases, MHBL would like to develop the Doris North Project in such a way that the site disturbance is minimized, and that the total capital investment is not disproportionate to the current Project scope. Therefore, the designs presented in this report are somewhat unconventional as compared to other gold mines, although no compromises are made with respect to safety or environmental protection.

3.2 Foundation Conditions

3.2.1 Geotechnical Investigations

Site specific foundation investigations have been carried out at the Doris North site to identify foundation conditions. These site investigations can be summarized as follows:

- EBA completed six onshore and four offshore drill holes in Roberts Bay in 1997, about 2 km northwest of the jetty location selected for the Doris North Project. Two thermistor strings were installed in onshore holes (EBA 1997).
- SRK completed two drill holes in September 2002 in the vicinity of the mill site (SRK 2005c).
- Fifteen drill holes were installed in the winter of 2003, and included holes at the mill site, the beach laydown area, the airstrip location, the explosives storage facility location, along the secondary and primary road alignments as well as adjacent Doris Creek where the bridge abutments are to be constructed. Eight of these holes were completed with thermistor installations (SRK 2005c).
- Four drill holes were completed through the sea-ice in Roberts Bay at the Jetty location in the winter of 2004 SRK (2005d).
- A Nilcon vane-shear apparatus was used to test in-situ shear strength of the jetty foundation soils through the sea-ice in April 2005. A total of six borings was completed as part of this investigation (SRK 2005e).
- In April 2006 seven additional drill holes was completed at the jetty to obtain in-situ samples of the marine soils for strength characterization. This work is summarized in SRK (2006a), which has been included as Appendix B to this report.

In addition to these geotechnical investigations specifically focussed on the surface infrastructure components, there has also been a large number of characterization drill holes carried out for the design of the TCA, as documented in SRK (2007a). The foundation conditions within the TCA boundaries appear identical to those under the surface infrastructure components (with the exception of the jetty), and therefore all the information is relevant. Dwg. G-04 presents a complete summary of all geotechnical drill holes completed at the Doris North site.

3.2.2 Jetty

The jetty will be about 103 m long, and will, for the most part, be in water less than 2 m deep; however, the jetty terminus will be in water over 5 m deep. Sea ice approximately 2 m thick develops in the winter and freezes to the bottom of the seabed for at least the first 55 m of the jetty. The drilling results confirm sub-ocean permafrost to at least this location, consisting of a 3 to 5 m thick layer of frozen marine silt and clay over 6 to 9 m of sand and gravel.

The remainder of the jetty foundation consist of unfrozen marine silt and clay, 8 to 12 m thick. In-situ and laboratory strength testing confirms that the marine sediments have peak shear strengths between 14 and 28 kPa, increasing gradually in strength with depth. These overburden soils overlie competent basalt bedrock.

3.2.3 Other Areas

The onshore surface infrastructure foundation soils generally consist of sand, marine silts and clays of medium plasticity, ranging between 5 and 15 m thick. Permafrost conditions are present everywhere, with ground ice generally about 25%. The marine clays tend to contain saline pore water, with salinities equivalent to that of sea water. The underlying bedrock is generally competent basalt.

3.3 Hydraulic Design

Storm water conveyance systems and facility sumps will be designed for the 1:100 year recurrence interval storm event of 24-hour duration. This event has a magnitude of 48.6 mm. Storm water and pollution control containment ponds will be designed for this storm event taking into consideration 6-hour pumping cycles, with one operational stand-by pump available at all times.

Snow removal from critical structures is part of the regular maintenance procedures, and therefore no specific design criteria for snow accumulation have been allowed in those cases. In areas where snow removal is not practical an allowance for the annual 65 mm of annual snow water equivalent has been allowed for.

3.4 Design Earthquake

The Doris North site is not seismically active, falling within the lowest seismic hazard rating category for Canada. No special seismic design standards have been adopted for the surface infrastructure components. All structural fill will be placed with angle of repose side slopes, which is about 40^{o} for the parent basalt bedrock.

3.5 Borrow Materials

Air photo interpretation, supported by the field investigations has confirmed that local marine clay silt and borrow deposits could be developed. The clays and silts would however not be of much use given the type of construction to be carried out, and although the sand could be useful as concrete aggregate, the complications associated with developing a soil borrow pit in a permafrost environment makes development of these sources undesirable.

Therefore, all construction materials for the Doris North Project will be crushed and processed material from one of four quarry sites identified on the site, as indicated on Dwg. G-02. The locations of these quarries have been based on ease of development and proximity to the construction areas. Geochemical analyses have confirmed that rock from each of these quarries would be suitable for use as a construction material, including concrete aggregate (SRK 2007d).

Development of the rock quarries does not require disturbance of the surrounding permafrost, with the exception of either winter road access, or an all-weather road. The rock quarries do not contain massive ice or ground ice and will be developed through conventional drill-and-blast techniques. The rock mass is competent, and conventional hard-rock bench design parameters is envisioned, consisting of 3 to 5 m high benches blasted with 80^o wall slopes. Bench setback will be between 3 to 5 m. These parameters will be adjusted as needed based on observed rock quality once quarry development starts.

During quarry development and operation, surface water management will consist of an upstream diversion berm to prevent runoff from outside of the quarry footprint from entering the area, as well as a downstream containment berm to contain surface runoff within the quarry footprint. The quarry base will have a low spot where this water can collect without affecting quarry operations.

3.6 Construction Materials

Nine different material classes have been defined for use in the Doris North Project. Grain size distribution envelopes for each of these classes are presented in Dwg. G-05. The material classes relevant to the designs presented in this report can be summarized as:

- Riprap – for use as erosion protection where needed.
- Run of quarry – for use as bulk fill.
- Select subgrade – for use as a filter or transition layer between the run of quarry and surfacing material.
- Surfacing material – for use as a traffic layer, or a liner protection layer.
- Drainage gravel – for use as a liner protection material.

Complete Specifications (SRK 2007b) regarding these materials are presented the in Project Technical Specifications document. All these materials will be produced on site at one or more locations. Appendix E contains preliminary summary of quarry material quantities for the Project.

Three synthetic products are listed to be used consistently with the natural construction materials. These are:

- 1.4 mm (57 mil) textured HDPE liner for containment in the fuel transfer station, fuel tank farm and the pollution control pond.
- 385 g/m^2 non-woven geotextile for protection of the HDPE liner. Appendix H contains details on the selection criteria adopted for the liner and geotextile.
- 40 x 40 mm bi-axial polypropylene geogrid as a structural support layer at the base of the jetty.

Since no low permeability material will be quarried, containment and diversion berms will be constructed with select subgrade material. This material in itself will be relatively permeable; however where these structures are constructed it is anticipated that the pre-screened portion of this material be used, which will contain sufficient fines to act as a suitable barrier. Notwithstanding this, it is expected that at the start of construction, a test berm will be constructed, and that if this berm is proven to be too permeable to perform its function, the design will be changed to include a liner or other suitable filter fabric.

3.7 Minimum Pad Thickness

Appendix D contains details of a thermal calculation that was carried out to determine the minimum granular pad fill thickness for the Doris North Project. For the most important structures such as the fuel tank farm and the mill and crusher foundations, a fill foundation will not suffice and these structures will be founded on competent bedrock. For less important structures such as the airstrip and the camp, the minimum pad thickness will be 2 m, whilst for regular structures such as the roads and other simple pads, the minimum fill thickness will be 1 m.

These pad thicknesses are not sufficiently thick that the active layer will necessarily remain completely within the fill material. Settlement of the pads and roads are therefore expected. The monitoring and maintenance procedures document in this report will ensure that such settlement can be managed adequately.

3.8 Diversion of Natural Drainage Paths

No natural streams or rivers will be diverted to accommodate mine infrastructure. Furthermore, none of the surface infrastructure (with the exception of the TCA) will interfere with any natural drainage system, fish bearing or otherwise, other than diffuse natural runoff. Water from upslope areas will be naturally diverted around the pads and road alignments. Ponding will be prevented by installing culverts at those locations where water would have been flowing seasonally if the infrastructure had not been in place.

The only permanent stream crossing affected by the Doris North Project is Doris Creek, and at that location a clear span bridge will be constructed.

4 Design of Surface Infrastructure Components

4.1 Jetty

Annual re-supply for the Project will be via barges from Hay River, NT. The barges typically arrive at Roberts Bay mid to late August every year. A jetty will be constructed, such that the barges can be directly offloaded without having to beach the barge. The jetty will therefore only be used for a period of two to three weeks every year. The jetty will only be used for the two years of mining, plus another two years of active decommissioning. Subsequent to active decommissioning the jetty will no longer be required, since further annual re-supply volumes are expected to be small and will be done via sealift to the existing barge landing site used for exploration in the Hope Bay Belt.

Due to the short design life of the jetty, and the complex foundation conditions, MHBL is satisfied to construct a jetty that will require annual maintenance. Therefore, the jetty has been designed to a substantially lower standard than would normally be required for a similar structure, and MHBL is prepared to accept the risks and consequences that those design criteria have. The risks include damage to the jetty due to large waves, storm surges and sea ice. Furthermore, annual settlement and frost heave could result in damage to the jetty. MHBL is however prepared to implement the necessary maintenance measures, to ensure safe operation of the jetty when the time requires.

The physical consequences of damage to the jetty include addition of construction rock and an increased jetty footprint. Operational consequences for these damages include delays to the offloading of the barges, with associated increased operational costs for the mine.

Design criteria for the jetty are summarized in Table 4.1, and detailed drawings of the jetty are presented as Dwg. J-01, J-02 and J-03. Appendix C contains detailed calculations for the jetty bearing capacity.

MHBL had committed to the NIRB that a new detailed bathymetric survey of the jetty area will be undertaken to determine whether there is sufficient water depth to allow the jetty to shortened. This survey was carried in the summer of 2006, and confirmed that the original design of a 103 m long jetty is appropriate. The complete bathymetric survey report is included as Appendix A.

Table 4.1: Summary of jetty design criteria

Design Component	Design Criteria
Vessel	Barge NT 1500 Series: 1,886 tonne dead weight; 76.2 m LOA; 17.1 m Beam; 0.97 m minimum freeboard; 3.05 m Draft
Vehicles	Integrated Tool Carrier (TC-28) = 11,412 kg Wheel Loader Komatsu WA500-3; operating weight = 31,000 kg (Supplied by NTCL for off-loading only (Provisions for unloading mill modules for the mine at the jetty have not been included; these modules will be offloaded at the existing barge landing site)
Tides	Tide levels in Melville Sound (north of the site), as listed below, are taken from Canadian Hydro-graphic Service Chart 7780. EHWL and ELWL are based on tides at Cambridge Bay. Tides are referenced to local Chart datum Extreme High Water Level (EHWL) = 0.5 m Higher High Water Level, Large Tide (HHWL) = 0.2 m Higher High Water Level, Mean Tide = 0.2 m Mean Water Level (MWL) = 0.0 m Lower Low Water Level, mean Tide = -0.1 m Lower Low Water Level, Large Tide (LLWL) = -0.1 m Extreme Low Water Level (ELWL) = -0.3 m These tide data are consistent with site specific data reported in Golder (2005) and Frontier Geosciences (2003)
Jetty Working Platform	Minimum Water Depth: Established to provide a minimum of 1 m keel offset for the Series 1500 barge below LLWL (i.e. minimum water depth of 4.05 m) Deck Height: Established to provide 1.0 m of freeboard above the HHWL
Roadway Width	6 m
Barge Ramp	Barges are supplied with a 7.5 m long ramp to span between the barge and the jetty structure. The maximum recommended grade of the ramp is 6%
Jetty Terminus Work Area	NTCL requires only 6 m of work space to offload the barges; however, they prefer a berthing face of at least 20 m wide. Barge unloading can be from barges orientated laterally or longitudinally to the jetty
Wave Conditions	Largest waves from North, with maximum wave height = 0.9 m Maximum sustained storm surge = 0.7 m
Geotechnical Parameters	Existing Seabed: Unfrozen and frozen Silt and Clay; Saturated unit weight = 18 kN/m^3; Peak Shear Strength = 15 kPa Existing Seabed: Frozen Sand and Gravel; Saturated unit weight = 18 kN/m^3 Engineered Fill: Rock fill; Unit weight = 19.62 kN/m^3
Mooring Hardware	Moorings are designed for a one day per month exceedance wind gust of 82 km/h; During periods of higher wind speeds, the barge shall not be at berth Mooring lines and hardware must manage individual loads of 30 tonnes

4.2 Roads, Turnouts and Caribou Crossings

All-weather roads are required to link the surface infrastructure components of the Doris North Project. Complete engineering drawings of the road alignments and profiles are provided in Drawings S-15 to S-26, and the road sections can be summarized as follows:

- North primary road (6 m wide and 1,179 m long); link between the jetty to the northern end of the airstrip (Dwg. S-15).
- South primary road (6 m wide and 2,400 m long); link between the southern end of the airstrip and the road junction leading to the tank farm and camp (Dwg. S16 and S-17).
- Secondary road (5 m wide and 5,470 m long); link between the South Dam and the junction leading to the float plane dock and the camp (Dwg. S-18 to S-20).
- Explosives storage facility access road (6 m wide and 306 m long); link between the south primary road and the explosives storage facility (Dwg. S-22).
- Landfill/landfarm access road (6 m wide and 143 m long); link between the south primary road and the landfill/landfarm (Dwg. S-22).
- Fuel tank farm access road (6 m wide and 171 m long); link between the south primary road and the fuel tank farm (Dwg. S-23).
- Camp access road (6 m wide and 548 m long); link between the south primary road and the camp and mill pads (Dwg. S-23).
- Portal access road (6 m wide and 506 m long); link between the camp pad and the portal (Dwg. S-24).
- Float plane dock access road (6 m wide for 265 m, and 20 m wide for 160 m); link between the float plane dock and the mill pad (Dwg. S-24).
- Decant access road (5 m wide and 378 m long); link between the secondary road and the decant location (Dwg. S-25).
- Tail Lake discharge access road (5 m wide and 191 m long); link between the secondary road and the tailings slurry discharge point (Dwg. S-25).
- Spillway access road (5 m wide and 65 m long); link between the secondary road and the North Dam (Dwg. S-26).

The roads will be constructed with a minimum fill thickness of 1 m to cover micro-relief and protect the permafrost. Typical roadway cross sections are illustrated in Dwg. S-17 and S-20. All roads have been designed for single lane traffic only. Road turnouts are strategically placed to allow safe passing. The largest design vehicle is a CAT-988 loader. Sections of the secondary road, as well as some of the access roads will share traffic with pipelines. No physical separation of the pipeline corridors has been provided. Primary road turnouts are 4 m wide and 30 m long (Dwg. S-11). Secondary turnouts are 7.5 m wide as illustrated in Dwg. T-13.

Caribou crossings will be provided at all road junctions, major bends and at regular intervals along stretches of the road where no junctions are present. The final locations of the crossings will be inspected by local elders and additional crossings will be added if deemed necessary. The crossings are 10 m wide and the approach ramps have a minimum grade of 5H:1V, as illustrated in Dwg. S-10.

4.3 Airstrip and Apron

The design aircraft are the De Havilland Twin Otter and the Dornier 228. The airstrip will be equipped with lights for night use and with instrumentation necessary to support Instrument Flight Rules (IFR) flights. Minimum airstrip design parameters were supplied by Arctic Sunwest and Summit Air in Yellowknife, NT.

The airstrip will be constructed by widening a 914 m long section of the primary road to 23 m (Dwg. S-03). The airstrip is not located in an optimum location with respect to the prevailing northwest winds. The charter companies have flown mock approaches to this airstrip and are satisfied that it would be safe to use, provided there is good visibility and light to moderate winds. MHBL is aware that the airstrip location as designed may result in plane and/or crew change delays.

A 40 m x 17 m apron for vehicle parking, an emergency power generator, fuel storage and emergency shelter will be constructed adjacent to the southern end of the airstrip. This apron will also provide ample room to allow the design aircraft to turn around. A similar apron is not required at the north end of the runway, as both design aircraft are capable of making a 180^{o} turn on the airstrip.

A typical cross-section through the airstrip, as well as a longitudinal profile is illustrated in Dwg. S-03. The minimum gradient along the airstrip is 0%, while maximum gradient is 1.5%.

Transport Canada (1993) requires the following lights and instrumentation for an airstrip of this size to allow night time use with a non-precision approach system:

- One illuminated wind direction indicator with a maximum height of 7.5 m. The wind direction indicator will be centrally located along the longitudinal dimension of the airstrip, approximately 60 m from the edge of the airstrip.
- A simple Approach Lighting System (ALS) consisting of a minimum of five white lights, 90 m apart, installed on the extended centerline of the airstrip over 450 m on each end and two lights abeam of the airstrip threshold.
- Low intensity white airstrip edge lights, in two parallel rows equidistant from the centerline, uniformly spaced no greater than 60 m apart.
- Airstrip threshold lights consisting of six white lights at each end, configured to operate with the airstrip edge lights.
- Airstrip end lights consisting of two groups of three red lights each, mounted in a line at right angles to the airstrip axis, not more than 3 m from the end of the airstrip.

If necessary MHBL will consider constructing a winter airstrip on Doris Lake every year capable of landing a Lockheed C-130/L-100 Hercules. This airstrip will also require lights and IFR instrumentation. This airstrip will be designed and constructed by a specialist contractor, according to guidelines supplied by First Air in Yellowknife, NT. Details pertaining to the construction, operation and maintenance of the winter airstrip are provided in the Main Water Licence Application Document.

4.4 Culverts

The surface infrastructure requires no permanent stream crossings, and therefore all culverts will only convey seasonal melt or rainwater. The culverts are expected to be operational as long as the roads are used, and are installed at low points along the road and at specified seasonal drainage paths.

The culverts are 900 mm diameter corrugated steel pipes in compacted fill. Hydraulic design confirm that 600 mm diameter culverts are adequate (assuming a 24-hour duration, 1:100 year storm event of 48.6 mm; zero attenuation; 100% runoff and 12 ha catchment area); however, to allow easy access for installation of the steam pipes, the culvert sizes have been increased.

There shall be a minimum of 0.5 m fill cover over the culvert. The road surface shall be raised to accommodate this with a minimum 5H:1V approach ramp over the culverts. The culvert will extend a minimum 1 m beyond the toe of the slope of the fill, as illustrated in Dwg. S-11.

Steam will be used to thaw out the culverts prior to spring. To facilitate this, there will be a 51 mm outside diameter steel pipe laid inside along the longitudinal axis of the culvert. The steel pipe will have a 1.5 m stick-up vertically from the upstream end of the culvert for steam delivery.

4.5 Bridge and Bridge Abutments

Doris Creek is at least 7 m wide in the area where the secondary road must cross under normal flow conditions. Since Doris Creek is a fish bearing water body, and is defined as navigable water, the crossing will be by means of a free span bridge. A pre-fabricated modular steel bridge will be assembled on two rock fill abutments such that the minimum bridge deck height above Doris Creek will be 2.1 m. This is not a design requirement per se, but is the net effect of designing abutments that would ensure that the permafrost integrity is maintained. The stream bank-full width (i.e. the ordinary high water mark) of Doris Creek at the crossing location is about 15 m. This is therefore the minimum distance that the abutment toes can be apart. Appendix G provides complete details about the hydrotechnical assessment of the proposed Doris Creek bridge.

The bridge design vehicle is a fully loaded CAT-740 haul truck. The bridge will have a 75 tonne capacity and measure 7.3 m wide by 32 m long. A firm specializing in the design and manufacture of steel modular bridges was subcontracted to provide SRK with a suitable bridge design.

The bridge will rest on two pre-cast concrete sills, and will be anchored at the ends by two pre-cast concrete wing walls. These concrete members are also designed by the specialist firm. The concrete members will be founded on the rock fill abutments. In order to accommodate the bridge deck, the abutments will be 10 m wide, and the minimum fill thickness of the abutments beneath the concrete sills will be 2.5 m, to ensure the active layer remain within the fill material. Details pertaining to the abutment design are provided in Appendix F.

The approach ramps leading up to the bridge deck will have a maximum slope of 5H:1V and due to the height of the abutment, there will be guard rails along the entire ramp length. Dwgs. S-12a and S-12b provides more details of the design.

4.6 Beach Laydown Area

A laydown area for temporary storage of bulk ammonium nitrate, equipment and other supplies will be constructed 100 m inshore from the high-tide level immediately adjacent to the north primary road. This is a self-imposed buffer distance adopted by MHBL.

The pad has been sized based on an estimate of annual supplies provided by MHBL. The 100 m x 60 m pad will be at least 1 m thick, and will have a minimum grade of 0.5% to allow shedding of surface runoff. This is illustrated in Dwg. S-01 and S-02.

4.7 Pump House Pad

A 10 m x 10 m pump house pad, 1 m thick will be constructed immediately adjacent the secondary road, en route to the decant location. A building will be erected on this pad to house the control systems for the reclaim and decant water pipeline systems. The pump house shall have its own spill containment measures built in, primarily associated with fuel spills for pumps and motors.

4.8 Explosives Storage Facility

The explosives storage requirements for the Doris North Project consist of; (1) 38,000 kg of explosives, (2) 39,000 detonators, and (3) peak annual supply of bulk ammonium of 700,000 kg. The prefabricated on-site AN/FO mixing plant will produce a maximum amount 10,000 kg at any one time. For design purposes the total amount of mixed product was assumed to be 20,000 kg which includes the weight of mixed explosives and half the weight of ammonium nitrate in the mixing plant building.

MHBL will subcontract explosives storage, mixing, transportation and handling to an outside specialist contractor. SRK has prepared a foundation pad layout in accordance with the requirements stipulated by the specialist contractor. The layout satisfies the rules and regulations governing the storage and mixing of explosives (NRCan 1995). The minimum distance requirements for the explosives storage facility are as follows:

- D1 = not applicable
- D2 = 50 m
- D3 = not applicable
- D4 = 170 m
- D5 = not applicable
- D6 = not applicable

- D7 = 465 m
- D8 = not applicable

For the purposes of the minimum distance requirements, the primary road has been classified as a lightly travelled road based on the following:

- The road is used to haul goods from the annual sea-lift to the camp lay-down area (a couple of weeks every year)
- Fuel is only hauled along this road for a 2 week period every summer
- The road is used to transport personnel from the airstrip to the camp (3 to 4 scheduled flights a week)
- Explosives will be hauled to the mine along this roadway
- The transportation of explosives shall be in accordance with the Explosives Act.

The actual explosives and detonator magazines will be Type 4 prefabricated magazines, contained within sea cans. The mixing plant will also be a pre-manufactured facility contained within a sea can. These sea cans shall be installed on six 1 m thick rock fill pads, linked together with roads, as illustrated in Dwg. S-04. Barrier berms between pads provide the necessary safety buffers.

4.9 Fuel Transfer Station

The fuel storage demand for the project is 7.5 million litres per year, as defined by MHBL. Fuel will annually be shipped to site via barge. The barge company will supply a pump and floating fuel line to pump fuel to a shore manifold in the fuel transfer station. Fuel trucks will haul fuel from the fuel transfer station to the tank farm, operating round the clock for a two week period after the barge arrives.

The fuel transfer station will be located across the road from the beach laydown area (Dwg. S-01 and S-02). The station will consist of a lined containment facility measuring about 32 m by 16 m. The containment area has been sized to retain at least 110% of the capacity of the largest fuel truck at 40,000 L. In reality the containment volume is significantly larger, as a result of constructability restraints. Ramps for the fuel trucks will be located at opposite ends of the station, allowing for drive-through access. Access ramps will have a maximum grade of 5H:1V.

The bulk fill under the lined area will be at least 1 m thick. The containment area will be graded such that there is a single collection sump. Spill containment will be provided by an HDPE liner sandwiched between two geotextiles, and covered with a protective layer of surfacing material.

At any time when water is contained in the sump (i.e. surface runoff or melt water), it would be subjected to water quality testing, and if deemed clean would be pumped out onto the tundra. If this water is contaminated it would be pumped into a water truck and disposed of in Tail Lake.

4.10 Fuel Tank Farm

The tank farm has been designed in accordance with all appropriate standards and regulations, both with respect to containment requirements and minimum distances. The 7.5 million litres of fuel storage for the Project will be within five steel tanks each measuring 14.8 m in diameter and 9.8 m high (information supplied by MHBL).

Spill containment will be in a bermed and lined area. The minimum required containment capacity is 100% of the volume of the largest single fuel tank (1,500 m^3) plus 10% of cumulative volume of all additional tanks (600 m^3). A minimum containment berm height of 0.8 m is required to meet this criterion. The containment area must be sloped towards a single collection sump.

To minimize the risk of fuel spills, tanker trucks will be loaded and unloaded via a manifold located within the confines of the tank farm. An access ramp with a maximum grade of 5H:1V will be constructed for this purpose. The area where trucks will be parked for this purpose is limited to a 12 m wide x 15 m long section immediately leading off the ramp. Over this area the minimum fill over the liner will be increased to 0.3 m.

The tank farm must be constructed on a precision blasted bedrock surface, to completely remove any risk of foundation settlement. A nominal levelling layer of crushed material will be placed before proceeding with the spill containment layers. Spill containment will be provided by an HDPE liner sandwiched between two geotextiles, and covered with a protective layer of surfacing material. Dwg. S-05 and S-06 presents the facility design.

4.11 Mill and Camp Pads

Pads are required to house the ore stockpile, mill building, crusher building, mill reagent storage, power plant, light vehicle refuelling station, workshop, camp, kitchen, offices, dry, sewage treatment plant, potable water treatment plant, environmental laboratory and first aid station. The mill and crusher buildings cannot withstand any differential settlement, and as a result a precision blasted pad will be cut into bedrock to facilitate construction of these facilities. A nominal layer of surfacing material will be placed on the bedrock surface to provide a smooth and graded surface for civil construction. Based on surface topography, the final mill pad size (about 75 m x 250 m), will allow room over and above the mill and crusher, for the ore stockpile (about 10,000 tonnes, or 15 days of mill feed), the mill reagent storage area and the power plant. Dwg. S-07 and S-08 provides details of these areas.

All other facilities will be erected on a 2 m thick rock fill pad measuring about 60 m x 150 m. Both pads will be graded such that all surface water runoff and melt water will drain into the sedimentation pond located immediately downstream of the camp pad. The mill and crusher buildings will be covered and will have self contained sumps in the event of spills. The ore stockpile area will be graded to drain towards the pollution control pond immediately downstream of the temporary waste rock pile.

The construction and operation details of the mill surface civil facilities to be erected on the pads are not part of the SRK scope of work; however, for completeness a brief summary of the elements of these facilities as it pertains to this design is presented below:

Service complex (workshop): The workshop and storeroom (service complex) will measure about 30 m x 50 m. The storeroom will house tools and equipment required for service and maintenance of the underground fleet. The workshop floor will be a concrete bunded structure with sumps to collect spillage or wash water. These sumps will be emptied into the tailings feed line and pumped to Tail Lake.

Service complex (wash bay): Mining equipment and other surface and underground vehicles will be washed prior to maintenance in a dedicated wash bay to be located in the service complex. Wash water for this activity will come from the site’s fresh water supply tank. The wash bay will be equipped with a sump to collect the dirty water, and will also be equipped with a divider to allow light hydrocarbons to be collected from the surface using oil adsorbent materials. There will be provision to remove the mud through conventional settling which will be sent to the mill tailings pump box for co-disposal with the tailings solids. The wash bay system will be equipped with equipment to allow heavier hydrocarbons to be removed from the wastewater using cyclone action and to facilitate some recycle of the wash water.

Mill reagent storage area: Mill reagents will be shipped and stored in 6.1 m x 2.4 m sea containers. A storage area measuring about 45 m x 42 m will be provided immediately adjacent the mill building for storing 20 containers, single stacked, 2 m apart. The mill reagent storage area has been sized based on the number of containers required to supply the mill for one year, as provided by MHBL.

Mill laydown area: An additional laydown area for equipment and supplies will be provided on the mill pad.

Power supply: All mine power for the mill and camp pads will be generated on site using four diesel generators. The generators will be installed in a permanent building measuring approximately 384 m^2. This building will be constructed in close vicinity to the mill and crusher buildings on the mill pad. Remote power at the TCA, fuel transfer station and the airstrip will be by small local portable diesel generators.

Camp and first aid station: The 175-person camp will be a combination of skid mounted units linked together via Arctic Corridors. Accommodation will consist of single rooms with attached bathrooms. The kitchen and recreation facilities will be an additional five skid mounted modular units joined to the rest of the camp via the Arctic Corridor. The first aid station will be located in a separate modular unit connected to the rest of the facilities.

Offices, dry complex and environmental laboratory: The offices will consist of two combined modular units, the environmental laboratory will be one unit, and the dry complex will be four combined units. All these units will be linked to the main camp via Arctic Corridors.

Sewage treatment: Sewage treatment will consist of a modular packaged biological treatment plant that will be brought to site fully assembled within two skid mounted 12.2 m x 2.4 m containers. The treatment plant will have a treatment capacity of 68.6 m^3/day. The camp wastewater is collected in a grinder pump lift station and discharged to the solids settling tank, housed within the unit. Clarified raw sewage overflows to the equalizing tanks that feed the extended aeration bioreactors.

Each bioreactor consists of an aerated primary side and a clarifier cone. Wastewater will enter the primary aerated side and will be mixed with the existing water by means of a bubble aeration system. The clarification cone separates developed solids from the treated wastewater, allowing solids to settle back into the aerated side of the tank. This action reduces the amount of total solids and improves treatment. Treated effluent will be collected in a discharge/recycle tank for delivery into the tailings line.

Mine water supply: Potable water, fire suppression water and up to 67% of the mill water will be supplied from Doris Lake. A single insulated and heat traced 4” diameter HDPE pipeline will be used to pump water to storage tanks at the mill pad. Details of the fresh-water intake in Doris Lake are provided on Dwg. T-11 and T-12. Potable water will be treated in a packaged plant installed in a 12.2 m x 2.4 m container and will consist of sand filtration followed by ultra violet light and/or chlorination treatment.

4.12 Float Plane Dock

A float plane dock will be constructed at Doris Lake. The dock will be a pre-fabricated modular unit, designed and manufactured by a specialist contractor. The modular unit can be dragged on shore during freeze-up.

The dock has been designed to allow offloading of supplies from a Twin Otter plane using a Bobcat forklift. The plane requires 7.5 m of berthing face against the dock, and a minimum water depth of 1.5 m when fully laden. Based on the most recent Doris Lake bathymetry (Appendix A), the dock would have to be 25 m long to ensure compliance with the design plane berthing requirements. To ensure sufficient buoyancy of the dock, as well as a safe working platform, the minimum width will be 4 m. Dock buoyancy will be provided via sealed HDPE pontoons. The dock will be held in place via six permanently installed bollards. These bollards will be embedded in bedrock. Complete dock design details are provided on Dwg. S-09.

4.13 Temporary Waste Rock Pile Pad

Temporary storage for about 135,000 tonnes of waste rock is required. Some of the waste rock is potentially acid generating, and as a result MHBL will return it all as underground backfill. However, until the waste rock is used as backfill, a temporary waste rock pile will be constructed on a 1 m thick pad of clean run of quarry rock immediately downstream of the mill pad (Dwg. S-07 and S-08).

This will allow for easy dumping and reloading of any waste rock without a risk of damage to the underlying permafrost. Immediately downstream of the pad, a pollution control pond will be constructed to ensure complete containment of all surface runoff and melt water from the temporary waste rock pile and the ore stockpile. Berms strategically located along the edges of the pad will ensure complete containment, as well as prevent any clean water run-on of surface waters. Berms will be constructed with select subgrade material which has sufficient fines to ensure that water will be retained sufficiently long during storm events to allow diversion.

4.14 Pollution Control Pond

The pollution control pond is designed to contain all surface runoff and melt water from the ore stockpile and the temporary waste rock pile. The pond is designed for full containment of the 1:100 year storm event of 24-hour duration, plus an additional freeboard of 0.3 m.

Containment is provided, at least to the full supply level of 35.7 m by an HDPE liner sandwiched between two geotextiles (Dwg. S-07 and S-08). A protective cover layer is placed over the liner. No emergency spillway is provided, since it is intended that pumping out of this facility be initiated whenever there is at least one hour of pumping capacity in the pond. The pond pumps are designed to completely empty the pond within six hours.

4.15 Sedimentation Pond

The sedimentation pond is designed to retain all surface water runoff and melt water from the remaining areas of the mill and the entire camp pad. The hydraulic design criteria are identical to that for the pollution control pond.

The sedimentation pond is not lined (Dwg. S-07 and S-08). The run-of-quarry material that will be used to construct the pond will contain a significant amount of fines and the embankment retaining the water will be at least 8 m wide. Leakage is expected to be very low and as such allow ample retention time to ensure settling of solids. However, once the pond has been constructed the embankment will be inspected and if deemed too coarse, the upstream face of the dyke will be clad with a 12 oz geotextile to ensure appropriate retention time.

An emergency overflow is provided for the pond in the form of an outflow culvert located at the pond full supply level of 35.7 m. The overflow is provided such that the access road will not be overtopped during an extreme storm event.

4.16 Landfill

Non-combustible and non-hazardous waste will be disposed of in a landfill that will be constructed in a portion of the rock quarry immediately west of the camp (Quarry #2). This rock quarry will house both the landfill and landfarm (see next section). A surface area of at least 100 m x 100 m will be dedicated to the landfill as illustrated in Dwg. S-13 and S-14. In addition, any remaining surface area within the quarry that is not occupied by the landfarm will be used for landfill. Details regarding the expected waste volumes and types to be deposited in the landfill are provided in Supporting Document S10g – Landfill Management Plan. These estimates suggest that the total expected waste volume will be less than 1,000 m^3, and the landfill will have a capacity of at least 30,000 m^3.

The landfill will be completely hydrologically isolated via a set of perimeter containment and barrier berms as illustrated in Dwg. S-13. These berms will be constructed from select subgrade material and will be 1 m high (similar in design to the temporary waste rock pile containment berms depicted in Dwg. S-08). The landfill is also hydrogeologically isolated due to the presence of permafrost, and the host rock is of good quality such that cracks or fractures created by blasting is expected to be surficial and will not propagate any leachate.

The landfill surface will be graded towards a single sump (about 1 m x 1m x 0.5 m deep, for a containment volume of 500 litres) to allow collection and management of all flows. Complete details of this surface runoff management plan are provided in Supporting Document S10g. The landfill will be fenced and access will be via lockable vehicle gate.

Waste oil will be burned on site in a dedicated waste oil burner specifically designed for that purpose. Unused explosives will be burned or destroyed on site and unused chemicals as well as any other hazardous material will be disposed of in an appropriate manner.

4.17 Landfarm

The landfarm has been designed by others (AMEC 2006). SRK prepared engineering drawings in accordance with the information stipulated in AMEC (2006). The landfarm will be completely within the quarry development used for the landfill, as illustrated in Dwg. S-13 and S-14. Complete details regarding the estimated volumes of materials that may be handled in the landfarm facility, as well as the facility water balance is provided in Supporting Document S10h – Lanfarm Management Plan.

5 Operation and Maintenance Procedures

5.1 Jetty

5.1.1 Operation

Operation of the jetty will be according to the following procedures:

- As soon as the sea-ice is melted, the jetty will be inspected, and the annual maintenance to be carried out on the jetty will be defined and carried out in anticipation of the barge arrival in mid to late August (see next section for a description of what this maintenance will entail).
- Carry out a bathymetric survey of the approach channel to the jetty terminus, as confirmation to the barge company that there are no hazards present.
- The blast mats which are used as fenders against the jetty terminus will be installed.
- The buoys housing the floating mooring lines will be reconnected and the floating mooring lines will be attached to the shore anchor blocks and the buoys.
- When the barges arrive they will be pushed alongside the jetty terminus by the tugboat and the mooring lines will be connected. The barges will normally be berthed abeam; however, should conditions require the barge can be berthed along its bow or stern.
- A ramp supplied by the barging company, will be deployed, and supplies will be offloaded onto the jetty terminus by a loader supplied by the barge company. MHBL will use their own loader to transport these supplies to the beach laydown area.
- Once the barges have left, the mooring lines, buoys and blasting mats will be retrieved and stored in the beach laydown area until the next operating season.
- If strong winds or large waves are present, barge offloading will be temporarily terminated.

MHBL will be the only official user of the jetty. MHBL does however acknowledge that local communities may make use of the jetty whilst it is in operation. Access to the jetty will not be restricted unless MHBL is of the opinion that the jetty is not safe to use.

5.1.2 Maintenance

It is expected that the jetty will continue to undergo differential settlement over its lifetime, although the rate of settlement will likely exponentially decrease as time progresses. Considering the fact that the jetty will only be in use for two to three weeks in any year, this differential settlement can be managed with a program of annual maintenance that will consist of the following:

- As soon as the sea-ice has melted, the jetty surface will be surveyed and visually inspected by a qualified person, experienced with the design and intent of the jetty. The results of the survey and visual inspection will be used to determine how much settlement has occurred, and how much new rock fill would have to be added to return the jetty to its design operating standard (i.e. maintaining a safe trafficking surface with at least 1 m freeboard above the HHWL).

- For planning purposes, SRK recommends stockpiling sufficient additional fill material for about 0.5 m settlement every year, for the four year design life of the jetty. This amounts to a rock fill allowance of about 350 m^3 per year.
- Depending on the location and quantity of maintenance fill placement every year, silt curtains may have to be deployed around the construction area to ensure containment of suspended sediments that may be mobilized as a result of fill ravelling down onto the soft marine foundation.

In addition to the settlement maintenance described above, there are a number of other routine maintenance components for the jetty:

- Every year, prior to the arrival of the barge, the barge operator requires that a bathymetric survey of the channel leading up to the jetty terminus be carried out, to ensure that there are no sub ocean hazards.
- The coverage of this bathymetric survey should be extended to include the jetty footprint, since this data will be useful in tracking the settlement progress, as well as provide advance warning if there are any shoreline processes which may affect the jetty performance.
- All mooring hardware must be inspected prior to the arrival of the barge, and any equipment that show signs of wear, damage or corrosion must be replaced or repaired.

5.2 Roads, Turnouts and Caribou Crossings

5.2.1 Operation

Generally operation of the roads, turnouts and caribou crossings does not require any special consideration, outside of what would normally be considered applicable from a safety perspective. MHBL will set road and traffic rules, and will enforce these as necessary. All roads are for the sole use of MHBL; however, MHBL acknowledges that the roads will be used by the local communities if and when they pass through the area. Some specific operational issues are as follows:

- Considering the fact that the road design does not include a safety berm, MHBL must take all necessary precautions to post appropriate warning signs along the roads to advise road users of any potential hazards along the way. Strict enforcement of the speed limit is also recommended.
- If necessary, MHBL may apply water to the road surfaces in the summer months as a dust suppression agent. The water will be drawn directly from Doris Lake, and will be deployed by a tanker truck. No chemical dust suppressants will be used.
- Winter snow clearing will be done using a snow cat, or some other suitable equipment. The snow will be pushed off the side of the road, always towards the downstream side where practical. Care must be taken not to block culverts or instrument clusters.

- Generally no winter de-icing agents will be used. If ice makes the roads impassable, friction methods will be used such as application of pea-gravel as opposed to application of salt.
- Many of the roads share space with pipelines. MHBL should take all necessary precautions to ensure that road users are aware of where these pipelines are at all times.

5.2.2 Maintenance

Road maintenance will be an ongoing task, and will consist of the following components:

- During the summer months, the road and turnout surfaces, as well as the caribou crossings must be regularly visually inspected for signs of settlement, potholes, ruts or any standing water. Should any of these signs be detected, maintenance should be carried out using a conventional road grader using standard road grading procedures for gravel topped roads. The grader must first roughen up the surface, re-shape the crown and remove any ruts and/or potholes. Periodically new topping gravel may have to be placed on the surface to fill in voids such as potholes or undue settlement, or to re-shape the road crown. MHBL should prepare stockpiles of surfacing material expressly for this purpose during the initial construction phase.
- Winter road maintenance entails predominantly snow removal. Snow removal must be done with due care to avoid removal of any road surfacing material with the snow. Stockpiling of snow must be done in such a fashion that no large ponds will be created during the spring melt. The caribou crossings do not have to be cleared of snow; however, snow removed from the roads may not be stockpiled on the caribou crossings.
- During all maintenance activity, MHBL will have to take special precautions to ensure that the pipelines sharing some of the road alignments are not damaged.

5.3 Airstrip and Apron

5.3.1 Operation

The airstrip will primarily be used for crew changes; however, some equipment re-supply will also be done by air. Operating procedures associated with the airstrip are as follows:

- Since the airstrip doubles as the main road between the jetty and the camp, it will be necessary to stop all road traffic when aircraft are on the airstrip. MHBL will develop and put in place a protocol to manage this aspect.
- During winter months the airstrip must be cleared of snow prior to any landing taking place. No stockpiling of snow is allowed on the airstrip or apron, and should where practical be on the downstream side of the airstrip and not be higher than the airstrip grade.
- Routine checks must be carried out to ensure that the approach lighting system (ALS) is in complete working condition.
- Routine checks must be carried out to ensure that the IFR instrumentation has valid calibration certificates, and are in working condition.

- Depending on re-supply requirements, MHBL may construct a winter airstrip on Doris Lake. The design and construction of such an airstrip will be done by, and under the supervision of a specialist contractor. The size of the airstrip will be dependant on the re-supply needs, but may include landing of fully laden Hercules C-130 aircraft.
- All aircraft landing at the site will be charter planes, and will have a quick turnaround time. Therefore no de-icing equipment will be provided on site.

5.3.2 Maintenance

Airstrip and apron maintenance will be similar to that required for the roads. One additional maintenance requirement however entails continuous inspection of the ALS. MHBL must take all necessary precautions to identify the locations of all lights associated with the ALS, and special care must be taken so as not to damage these lights during snow removal or summer surface levelling and repair.

5.4 Culverts

5.4.1 Operation

There are no culverts at the Doris North site that allow passage of permanent streams, and there is no permanent aquatic life present at any of the culvert locations. All culverts will only have flow during the spring thaw, and possibly during heavy precipitation events.

The only period when any work is required on the culverts are at the onset of spring. At that time snow must be removed from the up and downstream end of each culvert opening and each culvert must be physically inspected to confirm if there is any indication of ice blockage. If ice blockage is present, steam will be used to thaw the culvert.

5.4.2 Maintenance

Maintenance of the culverts is limited to the following:

- Regular visual inspection to ensure that there are no objects that would obstruct the free flow of water through the culverts.
- If any culvert settlement has occurred, the amount of settlement must be documented and if necessary the culvert will have to be excavated, removed and re-installed to the original invert level after backfilling the settlement void with a competent material.
- If any wild life takes up habitat in a culvert, the animal will have to be relocated by appropriate specialists.

5.5 Simple Pads (Camp, Mill, Beach Laydown Area, Explosives Storage Facility, Pump House Pad and Temporary Waste Rock Pile Pad)

5.5.1 Camp and Mill Pads Operation

The camp and mill pads will house all the processing and accommodation facilities for the Project. As far as the geotechnical design is concerned, there are no special operating procedures associated with these pads, other than snow clearing. From a water management aspect the following operational procedures apply:

- The mill, crusher and workshop will be in individually enclosed buildings and will have self contained sumps to contain any spills. Emptying of these sumps will depend on the nature of the spill, and may be returned to the mill, be pumped to Tail Lake, or even send to the landfarm.
- The ore stockpile and the temporary waste rock pile are considered to be “dirty” water areas, and all runoff from these locations will be directed to the pollution control pond. Water in this pond must be tested, and if deemed unsuitable for general discharge, it will be pumped to Tail Lake.
- The remaining surface areas of the camp and mill pads are generally considered “clean” surfaces; however, all runoff and melt water from these pads will be collected in the sedimentation pond. Once suspended matter has settled, and if the water quality in the pond is deemed acceptable, this water will be pumped out onto the tundra. If the water quality is not deemed to be of acceptable quality it will either be used as mill make-up water or pumped to Tail Lake.

5.5.2 Beach Laydown Area Operation

Prior to arrival of the re-supply barges, the beach laydown area will be cleared as much as practical by transporting all surplus materials to the mill laydown area. This is to provide sufficient room for storage of the new products. Winter snow clearing of the beach laydown area is recommended in case supplies have to be retrieved.

5.5.3 Explosives Storage Facility Operation

Two types of explosives will be used on site; (1) stick explosives, and (2) AN/FO. The stick explosives, detonator cords and detonators will be stored in pre-fabricated magazines, placed directly on the fill foundations. The AN/FO will be mixed on site in a plant housed in a sea can. Bulk Ammonium Nitrate in one tonne tote bags will be mixed with diesel fuel to manufacture AN/FO prills in 25 kg bags. Explosives manufacturing will be done in batch form, based on demand, typically manufacturing enough explosives for a three week period at any one time.

Explosives manufacturing will be done by an appropriately qualified and certified outside contractor, in accordance with all relevant federal, territorial and local laws and regulations.

5.5.4 Pump House Pad Operation

The pump house pad and the building erected on it require no special operating procedures, other than snow clearing.

5.5.5 Temporary Waste Rock Pile Pad Operation

The purpose of the temporary waste rock pile pad is to preserve the permafrost such that waste rock can be stockpiled and reloaded during any season of the year. Therefore, other than ensuring that the section of the pile receiving waste rock is clear of snow, there are no special operating requirements for this pad.

5.5.6 Temporary Waste Rock Pile Construction

Actual construction of the waste rock pile is not part of the surface infrastructure design scope of work; however, for completeness, the waste rock pile construction methodology will briefly summarized here to assist in understanding the design principles.

During mine development a peak of 135,000 tonnes of waste rock will require temporary storage, prior to all being returned underground (there is however capacity to store at least 200,000 tonnes of waste rock on the temporary pad). Total waste rock storage space will only be required for a period of 32 months. The waste rock pile will be constructed in lifts, each a maximum of 5 m high. Secondary lifts will not be benched. Pile side slopes will be angle of repose. Peak waste rock deposition rate will be approximately 545 tonnes per day.

The overall waste rock pile will be less than 50 m in height, it will contain less than 1 million tonnes of waste, it will have an overall compound slope of 40^o, and it will be constructed in lifts less than 25 m in height, at a rate significantly less than 25 m^3/linial metre of crest per day. Furthermore, the pile will be moderately confirmed by natural topography and will be constructed from strong and durable waste. Percolation of water through the dump is expected to be limited, since freezing in the dump will likely occur rapidly. Based on all these considerations, the temporary waste rock pile can be classified as being in Stability Class I, according to Table 5.2, page 70, in the British Columbia Mine Waste Rock Pile Research Committee's manual on Mined Rock and Overburden Piles (BCMWRPRC 1991). For such piles the failure hazard is classified as negligible and the design can be based on basic reconnaissance and baseline data such as is available for this site. No waste rock pile stability assessment has been completed for this reason.

5.5.7 Maintenance for Simple Pads

Maintenance for the simple pads discussed in the preceding sections is similar to that described for the roads. Where ruts, potholes or settlement areas are observed, they must be in filled or levelled using standard grading equipment. Special care must be taken to ensure that there are no areas of standing water on the pads, and especially under heated buildings, inspections must be carried out to confirm that the active layer thickness has not increased.

The highwall slopes of the mill pad must be inspected by a qualified person every year to ensure that the face remain stable. Special care must be taken to inspect the state of permafrost degradation (if any) that may have occurred in the overburden cut slopes at the top of the highwall. Any measures to ensure the stability of the slopes must be carried out as part of the regular site maintenance program, and may include rock bolting the highwall, or re-armouring the overburden slopes.

5.6 Fuel Transfer Station and Fuel Tank Farm

5.6.1 Operation

Fuel barges will be moored at the jetty, or possibly anchored some distance offshore. The barge company will supply a floating fuel line and a pump (located on the barge) to offload fuel. The floating fuel lines will be connected to a shoreline manifold inside the fuel transfer station. Fuel trucks will drive into the fuel transfer station and will be filled via the shoreline manifold. The fuel trucks will then transport the fuel to the fuel tank farm from where pumps will transfer the fuel to one of the five primary fuel tanks. This fuel transfer will continue around the clock for a period of about two weeks until the barges are empty.

Standard operating procedures associated with the fuel transfer station and the fuel tank farm is as follows:

- MHBL will develop and put in place a fuel transfer protocol, which will be strictly enforced.
- Both the fuel transfer station and the tank farm will be kept clear of any snow, ice or water throughout the year.
- Prior to pumping of any water from the sumps, the water shall be tested, and if there are any signs of contamination, the water will be pumped to the mill for re-use, or to Tail Lake for disposal.
- Regular visual inspections must be undertaken specifically focussing on the liner integrity. Should there be any signs of liner damage, all fuel transfer must be halted until the necessary liner repairs have been carried out.

5.6.2 Maintenance

Fuel transfer station and fuel tank farm maintenance can be summarized as follows:

- Prior to arrival of the fuel barge, all pumping hardware must be inspected, and any components that show signs of wear, damage or corrosion must be repaired or replaced.
- The fuel transfer station and the fuel tank farm must be completely cleared of any snow, ice or water.
- The facilities must be inspected, and if there are any signs of settlement that may put undue stress on the liner, the cover must be excavated, the liner over the settlement area must be cut away and the settlement must be filled in. The liner must then be repaired or replaced as appropriate, and the cover material replaced.

5.7 Float Plane Dock

5.7.1 Operation

The float plane dock operating procedures are as follows:

- The dock will only be used during the open water season.
- When freeze-up starts the dock will be dragged on shore and winterised for storage.
- When the lake ice has disappeared, the dock will be inspected before re-floating and attaching it to the permanent bollards.
- Once floating, the deck will be inspected to ensure that it is in safe working condition.
- Float planes will be able to berth against the dock along any of its three faces.
- Care must be taken when clearing snow from the dock access road, that the dock which has been stored for the winter is not damaged.

5.7.2 Maintenance

Dock maintenance entails a detailed pre-season inspection of all components of the dock to check for wear, damage or corrosion. Any such components must either be repaired or replaced prior to re-floating of the dock.

5.8 Bridge and Bridge Abutments

The bridge abutments are rock fill structures that will require the same operational and maintenance procedures as those listed for the roads. Some additional maintenance aspects of the bridge include:

- Annually, the bridge and the safety guard rails along the approach to the bridge must be thoroughly inspected for wear, damage and corrosion. All deficiencies must be replaced or repaired as necessary as soon as practical. If in the opinion of the inspector there is a safety concern, the bridge will be decommissioned until the repairs have been carried out.

5.9 Pollution Control and Sedimentation Ponds

5.9.1 Operation

The pollution control pond will capture all surface runoff and melt water from the ore stockpile and the temporary waste rock pile. Pumps will convey the water in this pond to Tail Lake, or return it to the mill for re-use. The sedimentation pond will capture all surface runoff and melt water from the remainder of the mill pad and the camp pad.

Both ponds have been sized to contain the 1:100 year, 24-hour duration storm event (assuming no flood attenuation and 100% runoff), plus 0.3 m of freeboard. Pumps will be sized to allow complete draining of the ponds in 6 hours.

Once there is sufficient water in the pollution control pond to allow at least 1 hour of continuous pumping, the pumps will be switched on. Water will be pumped from the sedimentation pond, only after there is visual evidence that any suspended matter has settled, and a water quality sample had been taken and analysed. However, no ponded water is allowed in the sedimentation pond for more than 48 hours at any given time. This is to ensure that the permafrost integrity is maintained.

5.9.2 Maintenance

Maintenance tasks for the two ponds are as follows:

- Immediately before freeze-up, both ponds must be pumped dry to allow sufficient storage capacity when the melt starts.
- Throughout the winter the ponds may remain snow filled; however, before freeze-up a path to the pumping sumps must be cleared.
- Regular visual inspections must be made specifically focussing on the liner integrity. Should there be any signs of liner damage, repairs must be carried out as soon as possible.

5.10 Landfill

5.10.1 Operation

Landfill operating procedures will be as follows:

- All non-hazardous garbage will be deposited in a “cell” of the landfill. The size of these cells will vary depending on the specific disposal needs.
- Over the summer period, as needed, and immediately prior to the winter period of snow accumulation open cells will be closed by covering them with a nominal layer of surfacing material. This material is intended to fulfill a separation function only, and is not intended to limit infiltration since the waste is expected to be inert.
- Wastes produced during the winter months will be stored in a designated portion of the landfill and then relocated into an operating landfill cell following the spring thaw.
- Wastes will be dumped in rows and covered as required. Wastes will be disposed of directly on the ground and compacted with heavy equipment against the berm or existing row.
- At all times surface runoff and melt water must be directed to a single sump from where the water will be managed in accordance with the surface runoff management plan stipulated in Supporting Document S10g.

5.10.2 Maintenance

Landfill maintenance items are as follows:

- Snow clearing.
- Visual inspection to confirm that the water containment and diversion berms are in working condition. Should there be any signs that the diversion and containment berms are not

retaining or diverting water as required, these facilities will be upgraded by suitable means such as lower permeability soils and/or installation of a liner.

- The highwall slopes of the camp pad must be inspected by a qualified person every year to ensure that the face remain stable. Special care must be taken to inspect the state of permafrost degradation (if any) that may have occurred in the overburden cut slopes at the top of the highwall. Any measures to ensure the stability of the slopes must be carried out as part of the regular site maintenance program, and may include rock bolting the highwall, or re-armouring the overburden slopes.
- The final landfill configuration, drainage pattern and slope stability of the final cover cannot be established at this time. This will be a function of exactly how much waste is produced and how the cells develop. At the appropriate time a detailed suitable design will be completed.

5.11 Landfarm

SRK produced engineering drawings of the landfarm in accordance with a design by AMEC (2006). Complete details regarding the operation and management of the facility is provided in Supporting Document S10h – Lanfarm Management Plan. This supporting document also contains details with respect to the estimated volumes of materials that may be handled in the landfarm facility, as well as the facility water balance.

6 Construction

6.1 General

With the exception of the jetty, the sensitive permafrost environment requires that all the surface infrastructure fill material be placed during the winter months when the ground is completely frozen. Once the primary foundation fill has been put in place construction of ancillary facilities on these pads is no longer season dependant, provided construction equipment does not have to go onto the tundra.

Mobilization of all construction equipment will be via barges from Hay River, NT. All equipment, plant, materials, fuel and explosives will be mobilized to Hay River by June 30, 2007 at the latest. The barges will arrive at Roberts Bay by mid to late August 2007, and the barges will be offloaded at the existing barge landing site used for exploration in the Belt. The Contractor will winterise the equipment in anticipation of the crew mobilization that will occur in December 2007, when construction will start.

This design report outlines the basic principles of construction; however, complete details are provided in the Technical Specifications (SRK 2007b) that should be read together with this report. These Specifications also contain the necessary information relating to the Quality Control and Quality Assurance (QC/QA) protocols that are to be followed for all aspects of construction.

6.2 Jetty

Construction of the continuous rock fill jetty will be carried out during the late summer open water season in Roberts Bay. Construction will however be suspended for a two week period in July whilst Arctic Char are migrating towards Little Roberts Creek to spawn. The basic components of the construction are as follows:

- Deployment of a silt fence around the entire jetty construction zone to ensure that any suspended sediments stirred up as a result of end-dumping quarry rock on the soft marine foundation can be contained.
- Two layers of geogrid will be placed on the seabed. These geogrids will extend at least 5 m beyond the outermost edge of the final jetty footprint and will be at least 5 m ahead of the current fill being placed. The geogrid overlap will not be less than 2 m. The placement of the geogrid will be done by qualified arctic divers.
- The jetty will be constructed from clean rock located in Quarry #1. The quarry rock will not be washed prior to placement. Since there will be some blast residue on this rock when it is placed in Roberts Bay, SRK modelled the water quality in the Bay to confirm that there would be no adverse environmental effects as a result of this practice. The results of this calculation are documented as an Appendix to SRK (2006f).

- Construction will consist of end-dumping the engineered fill from the shoreline towards the terminus of the jetty approximately 100 m offshore, directly onto the geogrid. After a few dump loads have been placed, a loader or dozer will be used to flatten the advancing front such that equipment can continue to end dump. In deeper water (more than 2 m depth) the initial rock fill be manually placed using an extended boom excavator. This will reduce the impact surcharge on the soft marine sediments and allow for more controlled placement of the fill.
- After completion of the bulk fill to the terminus of the jetty, the transition zone and jetty surfacing grade material will be placed once again moving from the shore advancing out towards the jetty terminus.
- Ancillary facilities such as the anchor blocks, mooring chains, shackles and blasting mats will be located and installed as construction commences.

6.3 Roads, Culverts, Turnouts and Caribou Crossings

Construction of all permanent roads will entail the following components:

- Clearing of the snow and ice off the road alignment immediately prior to fill placement.
- Construction fill will be placed by end-dumping along an advancing road surface. After end-dumping, the fill will be levelled with a dozer and subsequently compacted as per the Technical Specifications.
- The three types of fill making up the roads will be placed consecutively, with a new fill type only placed after the preceding layer has been completed to the design grade and level.
- Since the design road width is to narrow for duel lane traffic, the contractor will construct temporary rock fill road turnouts as construction progresses. These will be removed as construction advances.
- Culverts will be laid in place at designated locations and after selective placement of appropriate fill around the culverts, construction will advance normally. After road construction has been completed, the contractor will return to each culvert to install the steam pipes.
- Road turnouts and caribou crossings will be constructed using the same methodology as that used for the road construction.
- After completion of all road construction, road signs and guard rails will be installed as per the Technical Specifications.

6.4 Beach Laydown Area, Explosives Storage Facility and Pump House Pad

Construction of the 1 m thick pads for the beach laydown area, explosives storage facility and the pump house pad will be identical to that adopted for the roads. At the explosives storage facility there is a requirement to construct a containment berm around the AN/FO storage pad. This will be put in place after completion of the relevant pad using small shaping equipment such as a loader or excavator.

After completion of the pads, the ancillary facilities must be installed on the pads. These ancillary facilities include pre-fabricated magazines and buildings, which will be placed either directly onto the rock fill or onto timber blocks, as per the relevant manufacturer's recommendation.

6.5 Airstrip and Apron

Construction of the airstrip and apron will be similar to construction of the roads, with the only difference being that the fill thickness is twice as much. Upon completion of the airstrip and apron, airstrip lighting and instrumentation, as well as some other ancillary facilities will be installed according to the manufacturer's recommendations.

6.6 Camp Pad

Construction of the camp pad is identical to construction of the airstrip. On completion of the camp pad, ancillary facilities such as the camp complex and other portable buildings will be installed as per the manufacturer's recommendations.

6.7 Fuel Transfer Station

The construction components for the fuel transfer station are as follows:

- Preparation of the fill foundation will be similar to that used in road construction.
- The containment berms must be shaped and compacted using the appropriate small shaping equipment.
- Once the fuel transfer station foundation is completed, and liner tuck trenches have been excavated on the containment berms, the HDPE liner (sandwiched between to geotextile layers) that will form the primary containment will be installed as per the Technical Specifications.
- Finally, taking all the necessary precautions the liner protection fill will be put in place by end dumping and spreading with a loader.

6.8 Fuel Tank Farm

The fuel tank farm must be constructed on a bedrock foundation. The construction components are as follows:

- A level exposed bedrock bench will be prepared by drilling and blasting, taking due care that overbreak is reduced to a minimum.
- After the bedrock foundation has been prepared a nominal levelling layer of fill material will be placed.
- Containment berms, liner tuck trenches, liner installation and protective fill placement will be done using the same techniques as adopted for the fuel transfer station.

6.9 Mill Pad

The mill and crusher buildings must be founded on bedrock. Therefore the mill pad will be constructed by levelling a bedrock bench similar to the techniques mentioned for the fuel tank farm. There is however portions of the pad that do not require a bedrock foundation and those parts of the pad will be constructed as conventional rock fill, as per the procedures for the camp pad.

6.10 Temporary Waste Rock Pile Pad

Construction of the temporary waste rock pile pad is identical to that for the beach laydown area; with the exception that only run-of-quarry fill is required. Also, a nominal containment berm must be constructed along portions of the pad perimeter using small shaping equipment.

6.11 Bridge and Bridge Abutments

The components of the bridge construction are as follows:

- The abutments must be constructed using the same techniques as that used for the roads; however, the fill thickness is substantially greater, requiring more run-of-quarry fill material.
- Once the abutments have reached the elevation of the underside of the bridge deck system, the pre-cast concrete sills and retaining walls for the bridge must be put in place.
- The bridge must be assembled and lowered in place onto the sills using a crane. The contractor may not have construction equipment in the stream bed, unless they are working on a dedicated ice bridge.
- The remainder of the abutment fill must be placed up to the road deck elevation, taking care to use hand-compacting techniques adjacent to the retaining walls.

6.12 Pollution Control and Sedimentation Ponds

Construction of the pollution control and sedimentation ponds will use the same techniques as used for general fill placement and liner installation previously discussed.

6.13 Float Plane Dock

The construction components for the float plane dock are as follows:

- Using the lake-ice as a working platform, drill and install the six bollards for the float plane dock.
- The access ramp and float plane dock laydown area will be constructed using the same basic fill construction techniques used for the roads.
- The dock will be assembled according to the supplier’s specifications, and once the lake ice melts, the float plane dock will be lowered and anchored to the bollards.

6.14 Landfill

The landfill will be constructed in the developed Quarry #2. The final shape of the landfill will be determined once a quarry development plan has been prepared. The basic components of the landfill construction will however be as follows:

- The final quarry floor will be shaped and leveled such that any surface water will drain towards a single low point.
- Perimeter diversion and/or containment berms will be constructed around the effective area of the landfill using general fill placing techniques.
- A chain link fence will be erected on this berm according to the manufacturer’s specifications.

6.15 Landfarm

Within the confines of the landfill, an area will be demarcated for construction of a landfarm. The landfarm construction will follow the same techniques as that used for the fuel tank farm.

7 Monitoring and Instrumentation

7.1 Monitoring Requirements

The surface infrastructure components discussed in this report will require two types of monitoring:

- Visual monitoring – physical inspection of all fill surfaces taking special care to identify any areas that may have undergone settlement.
- Thermal monitoring – to evaluate the depth of the active zone, such that advance warning of potential settlement can be determined.

Further details pertaining to monitoring are provided in Supporting Document S10m – Monitoring and Follow-up Plan.

7.2 Thermistor Locations

A total of 8 thermistors have been installed in locations where surface infrastructure have been designed, as illustrated in Dwg. G-04. Where possible, these installations must be retained. In addition, new thermistors should be installed at the following locations as part of the fill construction:

- Jetty; two strings (Dwg. J-01)
- Fuel transfer station; one string (Dwg. S-01)
- Airstrip; three strings (Dwg. S-03)
- Camp pad; two strings (Not indicated on Dwg. S-07 since the final location can only be determined on site)
- Pollution control pond; one string (Not indicated on Dwg. S-07 since the final location can only be determined on site)
- Sedimentation pond; one string (Not indicated on Dwg. S-07 since the final location can only be determined on site)
- Float plane dock laydown area; one string (Not indicated on Dwg. S-07 since the final location can only be determined on site)
- Roads; five strings (Dwg. S-15, S-16; other locations are not indicated since the final location can only be determined on site)
- Bridge abutments; two strings (Dwg. S-12a)

Each of these stings should have at least three beads measuring between depths of 0.3 m and 3 m below natural ground surface. The thermistor strings need not have data loggers, but monitoring frequency of manual readings on all stings must be completed at least once a month. If warming trends are observed, this frequency should be increased as appropriate. These data should ideally be reviewed by a qualified geotechnical engineer, at least once a year to assist in making appropriate maintenance recommendations.

This report, "**Design of the Surface Infrastructure Components, Doris North Project, Hope Bay, Nunavut, Canada"**, has been prepared by SRK Consulting (Canada) Inc.

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8 References

Adams, J. and Halchuk, S. 2003. *Fourth generation seismic hazard maps of Canada: Values for over 650 Canadian localities for the 2005 National Building Code of Canada*. Geological Survey of Canada, Open File 4459, 155 p.

AMEC. 2003. *Meteorology and Hydrology Baseline, Doris North Project, Nunavut, Canada*. Report submitted to Miramar Hope Bay Ltd., August 2003.

AMEC. 2006. Landfarm Design and Management Manual, Doris North Gold Mine, Nunavut. Report submitted to Miramar Hope Bay Ltd., October 2006.

British Columbia Mine Waste Rock Pile Research Committee. 1991. *Mined Rock and Overburden Piles – Investigation and design Manual*. Interim Guidelines. May 1991

EBA Engineering Consultants Ltd. 1996. *Boston Gold Project, Surficial Geology and Permafrost Features*. Report submitted to Rescan Environmental Services Ltd.

EBA Engineering Consultants Ltd. 1997. *Boston Gold Project Geotechnical Investigation on Proposed Roberts Bay Port.* Report submitted to BHP World Minerals, October 1997.

Golder. 2001. *Report on Thermistor Data Review – Hope Bay Project*. Letter-report submitted to Miramar Mining Corporation.

Natural resources Canada (NRCan) 1995. *Quantity-Distance Principles: Users's Manual*. Explosives Branch, Department of Natural Resources, Government of Canada. Ottawa, ON. 87 p.

Rescan. 2001. *Supplemental Environmental Baseline Data Report, Hope Bay Belt Project*. Report submitted to Hope Bay Joint Venture.

SRK Consulting Inc. 2005a. *Preliminary Tailings Dam Design, Doris North Project, Hope Bay, Nunavut, Canada*. Report submitted to Miramar Hope Bay Ltd., October 2005.

SRK Consulting Inc. 2005b. *Groundwater Study, Doris North Project, Hope Bay, Nunavut, Canada*. Report submitted to Miramar Hope Bay Ltd., October 2005.

SRK Consulting Inc. 2005c. *Preliminary Surface Infrastructure Design, Doris North Project, Hope Bay, Nunavut, Canada*. Report submitted to Miramar Hope Bay Ltd., October 2005.

SRK Consulting Inc. 2005d. *Phase I Foundation Investigation, Proposed Roberts Bay Jetty Location, Doris North Project, Hope Bay, Nunavut, Canada*. Report submitted to Miramar Hope Bay Ltd., April 2004.

SRK Consulting Inc. 2005e. *Phase II Foundation Investigation, Proposed Roberts Bay Jetty Location, Doris North Project, Hope Bay, Nunavut, Canada*. Report submitted to Miramar Hope Bay Ltd., May 2005.

SRK Consulting Inc. 2007a. *Design of the Tailings Containment Area, Doris North Project, Hope Bay, Nunavut, Canada*. Report submitted to Miramar Hope Bay Ltd., October 2006.

SRK Consulting Inc. 2007b. *Technical Specifications for Tailings Containment Area and Surface Infrastructure Components, Doris North Project, Hope Bay, Nunavut, Canada*. Report submitted to Miramar Hope Bay Ltd., October 2006.

SRK Consulting Inc. 2007c. *Engineering Drawings for Tailings Containment Area and Surface Infrastructure Components, Doris North Project, Hope Bay, Nunavut, Canada*. Report submitted to Miramar Hope Bay Ltd., October 2006.

SRK Consulting Inc. 2006a. *Phase III Foundation Investigation, Proposed Roberts Bay Jetty Location, Doris North Project, Hope Bay, Nunavut, Canada*. Report submitted to Miramar Hope Bay Ltd., August 2006.

SRK Consulting Inc. 2007d. *Geochemical Characterization of Portal Development Rock, Doris North Project, Hope Bay, Nunavut, Canada*. Report submitted to Miramar Hope Bay Ltd., October 2006.

SRK Consulting Inc. 2007e. *Water Quality Model, Doris North Project, Hope Bay, Nunavut, Canada*. Report submitted to Miramar Hope Bay Ltd., October 2006.

Appendices

Appendix A

2006 Bathymetry (Golder Associates)

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REPORT ON

BATHYMETRIC SURVEYS
HOPE BAY PROJECT
HOPE BAY, NUNAVUT

Submitted to:

SRK Consulting Canada Inc.
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DISTRIBUTION:

6 Copies - SRK Consulting Canada Inc.
2 Copies - Golder Associates Ltd.

October 20, 2006 06-1419-007

EXECUTIVE SUMMARY

Golder Associates Ltd. (Golder) was retained by SRK Consulting Canada Inc. (SRK) to conduct bathymetric surveys for the proposed development of the Hope Bay Gold Project. This report is carried out in accordance with our proposal 06-1419-007, dated March 7, 2006. The field investigations were completed during a period extending from July 31 to August 29, 2006.

The objective of the site investigation was to provide single-beam bathymetric data on selected lakes in the area of the Hope Bay Project. Low-density bathymetric coverage was required on Doris, Windy, Patch, and Spyder Lakes and high-density coverage was required in Tail Lake, two areas of Roberts Bay, and approximately one-third of each of Windy, Patch and Spyder Lakes.

In particular, high-density information is required at specific areas of various lakes to aid the design of docking facilities, volume calculations and general mine design. The detailed bathymetric data can also provide a visual aid for the evaluation of potential faults and possible sediment flows. To complete this work single-beam bathymetry with real-time sub-metre positioning was used. In addition, low-resolution sidescan sonar imaging was observed during bathymetric fieldwork on selected lines for qualitative evaluation that the chosen density coverage was sufficient to map the terrain.

The bathymetry data provided good resolution of subsurface features. All of the lakes presented a non-uniform topography similar to surface topography in the areas. Many lineaments, including probable bedrock ridges are seen to extend into the lakes.

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LIST OF FIGURES

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by SRK Consulting Canada Inc. (SRK) to conduct bathymetric surveys for the proposed development of the Hope Bay Gold Project. This report presents the results from these investigations.

The objective of the site investigation was to provide single-beam bathymetric data on selected lakes in the area of the Hope Bay Project. Low-density bathymetric coverage was required on Doris, Windy, Patch, and Spyder Lakes and high-density coverage was required in Tail Lake, two areas of Roberts Bay, and approximately one-third of each of Windy, Patch and Spyder Lakes.

In particular, high-density information is required at specific areas of various lakes to aid the design of docking facilities, volume calculations and general mine design. The detailed bathymetric data can also provide a visual aid for the evaluation of potential faults and possible sediment flows.

2.0 SCOPE OF WORK

The proposed scope of work was as follows:

- General bathymetry of Doris, Windy, Patch and Spyder Lakes at 50-m line spacing;
- Detailed bathymetry (10-m line spacing) of approximately one-third of the survey areas of Windy, Patch and Spyder Lakes;
- Detailed bathymetry of Tail Lake;
- Detailed bathymetry of two areas within Roberts Bay;
- Global positioning system (GPS) positioning with 1-m to 2-m accuracy presented in NAD83 datum; and
- Preparation of bathymetric drawings based on supplied AutoCAD base maps.

In consultation with SRK, (the prime consultant to Miramar Hope Bay Ltd. for the mine design) the following techniques were selected to achieve the stated objectives in the survey area:

- Single-beam echo sounding;
- Low-resolution sidescan sonar to evaluate density coverage; and
- Real-time differential navigation utilizing the Canadian Differential Global Positioning System (CDGPS) with tide monitoring and tie-in to locally provided reference locations.

3.0 INSTRUMENTATION AND FIELD OPERATIONS

The surveys were carried out using a winterized Zodiac inflatable boat powered by a 15-horsepower outboard motor supplied by Miramar Hope Bay Ltd. This provided a lightweight boat that was moveable by helicopter and also provided relatively rapid surveying and good access and maneuverability to the frequent shallow areas encountered during the surveys.

The work was completed during the period July 31 to August 29, 2006. The sea state was generally good during data collection. As the survey progressed, the weather conditions deteriorated. In all, there were three days with no data collection due to adverse weather conditions affecting GPS quality, bathymetry accuracy (due to wave height) and safety. On these days, data processing and equipment maintenance was undertaken. The geophysical instruments and navigation system all operated within specification throughout the course of the entire survey. No reportable health and safety incidents occurred during the fieldwork.

The vessel position was acquired with a single-frequency code-based Trimble DGPS (Ag132) which in good GPS conditions can be accurate to approximately +/-0.5 m (see Section 3.2).

The bathymetry was measured using an ODOM® Hydrotrac Survey Echo Sounder with a 200-kHz transducer. This provided high-resolution bottom detection at a rate of 10 Hz. Velocity calibrations were completed at each of the lakes for accurate determination of sound velocity in water.

To ensure good coverage of lake-bottom features, especially for 50-m line spacing, we operated a low-resolution sidescan sonar during data collection. The sidescan sonar provides qualitative images (seismically) of bottom topographical variations to indicate whether additional bathymetric coverage should be completed.

The sidescan sonar was recorded using an Imagenex digital dual sidescan sonar (SportsScan). The SportScan utilizes two transducers of 330 kHz to provide a low resolution image of the lake bottom up to 60 m from the sensor in each direction. The data was recorded in conjunction with the GPS stream from the Ag 132 using the Imagenex software, Win881SS. The sonar was braced to the side of the boat at a depth of 1.2 m.

3.1 Survey Coverage

The boundaries of the survey were outlined and SRK requested coverage of all the selected lakes at a minimum of 50-m intervals. This line spacing ensured adequate coverage for volume calculations and identification of any unusual topographical features on the lake floors. Higher density areas were required in the following areas:

- the northern half of Patch Lake;
- the southern third of Windy Lake;
- all of Tail Lake and both Roberts Bay areas; and
- the western half of the Spyder Lake survey area.

Sidescan sonar data were obtained at Roberts Bay, Windy, Patch, and Tail Lakes. The survey lines were profiled on multiple traverses to provide a good overview of the lakebed features. Due to time constraints and adverse weather conditions, no sidescan data were obtained on Doris and Spyder Lakes.

3.2 Navigation

Positioning of the survey vessel and the sonar equipment was provided by Trimble Differential Global Positioning System (DGPS) receivers. Real-time corrections were obtained using industry-standard Canadian Differential GPS (CDGPS) corrections, and Wide-Area Augmentation System (WAAS) system as a backup. Vessel navigation data were acquired with a single-frequency code-based Trimble DGPS (Ag132) accurate to approximately +/-0.5 m. The navigation GPS antenna was installed directly above the bathymetry transducer to minimize offset errors. The onboard receiver provided differentially corrected WGS84 latitude and longitude values at 5 Hz to both the navigation computer and SportScan sonar.

Hypack Max software produced by Coastal Oceanographics was used for navigation. During the survey, the vessel position was continuously plotted on a chart showing the planned and actual survey lines. This information was displayed to the helmsman on a LCD monitor along with additional navigation parameters. The vessel position and single-beam bathymetric data were acquired digitally and stored on the navigation computer. Fix marks were recorded at 60-second intervals.

3.3 Datum and Tidal Corrections

At each of the sites, a stake was driven into the lake and water levels were recorded daily. Each of the stakes was surveyed by the on-site surveyor to the Miramar Hope Bay datum. All horizontal positioning was recorded internally as latitude and longitude using the WGS84 datum, then displayed as UTM Zone 13 coordinates using the NAD83 datum. All coordinates given in this report use the NAD83 datum, and UTM coordinates are plotted on the relevant deliverable figures.

Tidal corrections were obtained at Roberts Bay by observations of water levels noted on a wooden tidal post, placed in a sheltered cove at Area A. Our tidal measurements have confirmed that predicted tides from Canadian Hydrographic Service (CHS) models have similar phases and peak values to predicted tides. To convert the observed water level readings to the mine datum, the tidal post was surveyed in by Miramar Hope Bay surveyors.

4.0 RESULTS AND INTERPRETATION

This section summarizes the results of the bathymetric surveys. The data coverage and the interpreted bathymetry data are presented in Figures 2 to 8, in combination with the land topography. The water depths are contoured to 1-m intervals and blue-shaded to enhance visualization. The actual survey tracklines are presented on Figures 9 through 15. All figures are provided in electronic format on CD and were provided on an FTP site for downloading. The bathymetric data are also provided on CDs contained in each hardcopy report.

4.1 Positioning

Due to continued excellent satellite coverage, the Trimble DGPS positioning equipment provided high quality location fixes continuously throughout the surveys.

Real-time CDGPS corrections provided differential correction during most of the survey. Occasional loss of this differential signal occurred during the survey, due to rough water conditions or blocking of the satellite signal behind nearby topographical highs. During these periods, the system was set to utilize the WAAS corrections which still provided sub-metre differential corrections.

In post-processing, the navigation data are automatically filtered for any non-differential, high Horizontal Dilution of Precision (HDOP), or anomalous GPS data. This occurred in rare cases but not for any long time periods. When weather conditions were too rough to reliably gain a differential fix, a standby day was required.

The position in NAD83 coordinates and water elevation (at the time of surveying) of each of the survey stakes as provided by Miramar Hope Bay Ltd. are summarized below:

Survey Area	Easting	Northing	Elevation (m)
Patch	433893.3	7552217.8	26.28
Windy	432569.5	7550525.0	18.24
Doris	433800.0	7559050.0	21.42
Roberts Bay	432221.7	7563305.5	temporary mark = 0.92 m
Spyder	441135.5	7505824.0	65.63
Tail	435263.0	7557635.5	28.12

To record the tidal fluctuations at Roberts Bay, a stake was placed in the shallows at the coordinates mentioned above. A temporary depth scale was drawn on the stake and referenced each hour whilst surveying. The surveyors then calculated a true elevation for the temporary scale marked on the stake. True tidal elevations were calculated using the corrected information.

4.2 Bathymetric Results

The single-beam bathymetric data were of high quality and provide reliable depth data for the required lakes and ocean areas. The data have been combined, filtered, and contoured using AutoCAD and Surfer by Golden software.

The bathymetric results are presented in Figures 2 to 8 and have been provided in electronic format to SRK for incorporation into engineering drawings. For interpretation and planning purposes, we have also combined the bathymetric data with land topographical data that were provided by Miramar Hope Bay Ltd. through SRK.

Post-processing of the data included tidal corrections, removal of outliers and erroneous GPS positions.

4.2.1 Roberts Bay

The two areas within Roberts Bay were surveyed over two days during extremely calm weather, which provided reliable data and consistent tidal matches between days. Area A bathymetry data (Figure 2) indicate that water depths gradually increase to more than 7 m at the mouth of the cove. A shallow gradient shelf extends from shoreline to approximately 100 m into the cove. The water depths deepen rapidly from 3 m to 6 m at the edge of this shelf. The eastern side of Area A indicates a very shallow area which limited surveying due to insufficient draft for boat operation.

The data from Area B (Figure 3) shows the sea floor topography to be consistent with the shoreline trend. The near-shore area is characterized by shallow gradients. At 3 m depth, the gradient increases and depths increase to beyond 13 m at the edge of the survey area. Both areas within Roberts Bay were surveyed in a grid with a 20-m line spacing.

4.2.2 Doris Lake

The Doris Lake data (Figure 4) indicate water depths ranging up to 20 m. Notable features are a steep cliff at shoreline on the eastern shore of the lake which deepens to more than 16 m within a few metres from shore. The southern third of the lake is characterized by a relatively flat, shallow lake bottom, with depths not in excess of 6 m. Doris lake was surveyed at 50-m line spacing.

4.2.3 Tail Lake

Tail Lake (Figure 5) was surveyed at 10-m line spacing and indicates water depths of up to 7 m. Two north-south channels are present in the centre of the lake which are both approximately 1.5 m deeper than the surrounding area.

4.2.4 Windy Lake

The Windy Lake data (Figure 6) indicates water depths in excess of 22 m at a deep bowl located in the area of 431400E, 7553900N. An isolated shallow ridge occurs in the centre of Windy Lake with depths slightly less than 5 m encountered. The southern third of the lake was surveyed at 10-m line spacing, and indicates a gradual shoaling of water depth to the south with no major anomalies.

4.2.5 Patch Lake

The Patch Lake data (Figure 7) indicate a shallow lake of approximately 5 m in depth with three significant deep bowls of up to 16 m in depth. These depressions are indicated by the darker colours on the contour plan. The northern half of Patch Lake was surveyed at 10-m line spacing which delineated a number of smaller features such as a steep cliff down to 6 m in depth, located at 434500E, 7550800N.

A smaller lake (centered on 433700E, 7551400N) was attempted on three separate occasions. However, no GPS lock could be gained, due to the large cliff on the southwest shoreline obstructing the view of the satellites. This effect was also noticed in the northernmost area of Patch Lake where CDGPS correction could not be gained and the WAAS system was exclusively used. The depths observed in the small lake were all less than 4 m and a shallow area of under 1 m in depth occurs at the northeastern shore. Unfortunately due to lack of GPS signal, we did not record any data at this lake.

4.2.6 Spyder Lake

The Spyder Lake data shown on Figure 8 indicate water depths up to 19 m. The western half of the survey area reveals a deep, irregular channel which was surveyed with a line spacing of 25 m to provide extra delineation of the features. The eastern half of the survey area is generally flat with water depths of less than 5 m. Due to extremely low water conditions at the time of surveying, a few areas were too shallow and could not be surveyed. This includes; south of 7503300N and the small inlet near camp, centered at 441600E, 7505600N. A shallow reef is present at 440600E, 7505700N which broke the surface at the time of surveying and may also present a hazard to boats during times of higher water levels.

5.0 DISCUSSION AND SUMMARY OF INTERPRETED RESULTS

The bathymetry data provides accurate resolution of the underwater topography, especially in the high-resolution areas where 10-m line spacing was undertaken. The GPS data was consistently within sub-metre accuracy and multiple velocity calibrations were completed at each site to ensure using accurate sound velocity values.

Low-resolution sidescan sonar was conducted whilst surveying to help identify any major anomalies or highly variable lake bottom, that would require additional survey lines. This data was reviewed at the end of each day to ensure adequate coverage at the time of surveying. In general, this data presented few reflectors and anomalies in the centre of the lakes and significant boulders in near-shore areas.

All of the surveying was undertaken during a particularly dry summer which produced low water levels, especially in the case of Spyder Lake. We note that Spyder Lake at the time of surveying had an elevation of 65.63 m (approximately 1.5 m lower than springtime water levels) which resulted in many drill casings being partially exposed creating a safety hazard. The low water levels created problems entering certain bays (Figure 8) and also slowed survey progress due to frequent shallows. If more detail is required in these areas, it is recommended to conduct extra bathymetry during the high-water levels in the springtime or alternatively conduct an over-ice program, utilizing ground-penetrating radar.

In general, the strike and shape of the lake bottom topography reflects lineaments present on land, which may aid visual interpretation of faults and possible landslides.

No specific sediment or rock information can be gained from the bathymetry data. However, shape and gradient of slope may be useful in identifying areas of possible bedrock exposure. In all of the shallow areas encountered during the survey where bottom characteristics could be viewed by field personnel, the lake bottom consisted of soft silts interspersed with medium-sized boulders.

6.0 CLOSURE

This report has been prepared based on the information obtained for the purposes outlined above.

We trust that this report meets your immediate requirements. Please contact the undersigned should you have any questions or concerns.

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED BY

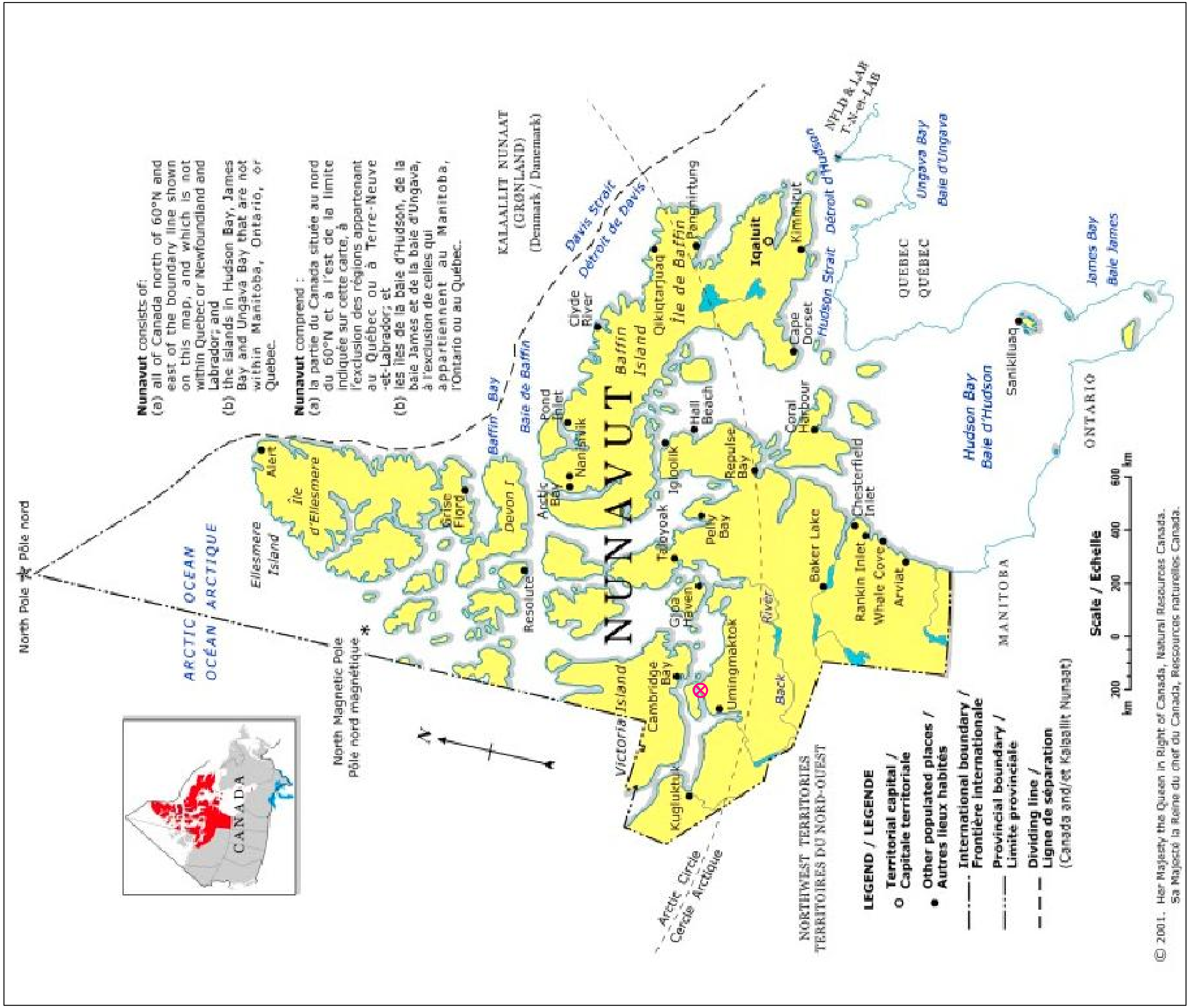
John Woods, E.I.T.
Geophysicist

ORIGINAL SIGNED BY

Michael Maxwell, Ph.D.
Senior Geophysicist, Principal

JKW/MGM/vee
06-1419-007
N:\FINAL\2006\1419\06-1419-007\RPT1020_06 - SRK - HOPE BAY BATHYMETRY.DOC

Drawing: O:\Active_2006\1419\06-1419-007 Hope Bay Bathymetry SRK\9-Cad\061419007_location.dwg Plot: 2006/10/20, 14:39 By: ntaylor



Reference:

Location image provided by SRK.

Legend:

⊗ Survey Area Location

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT

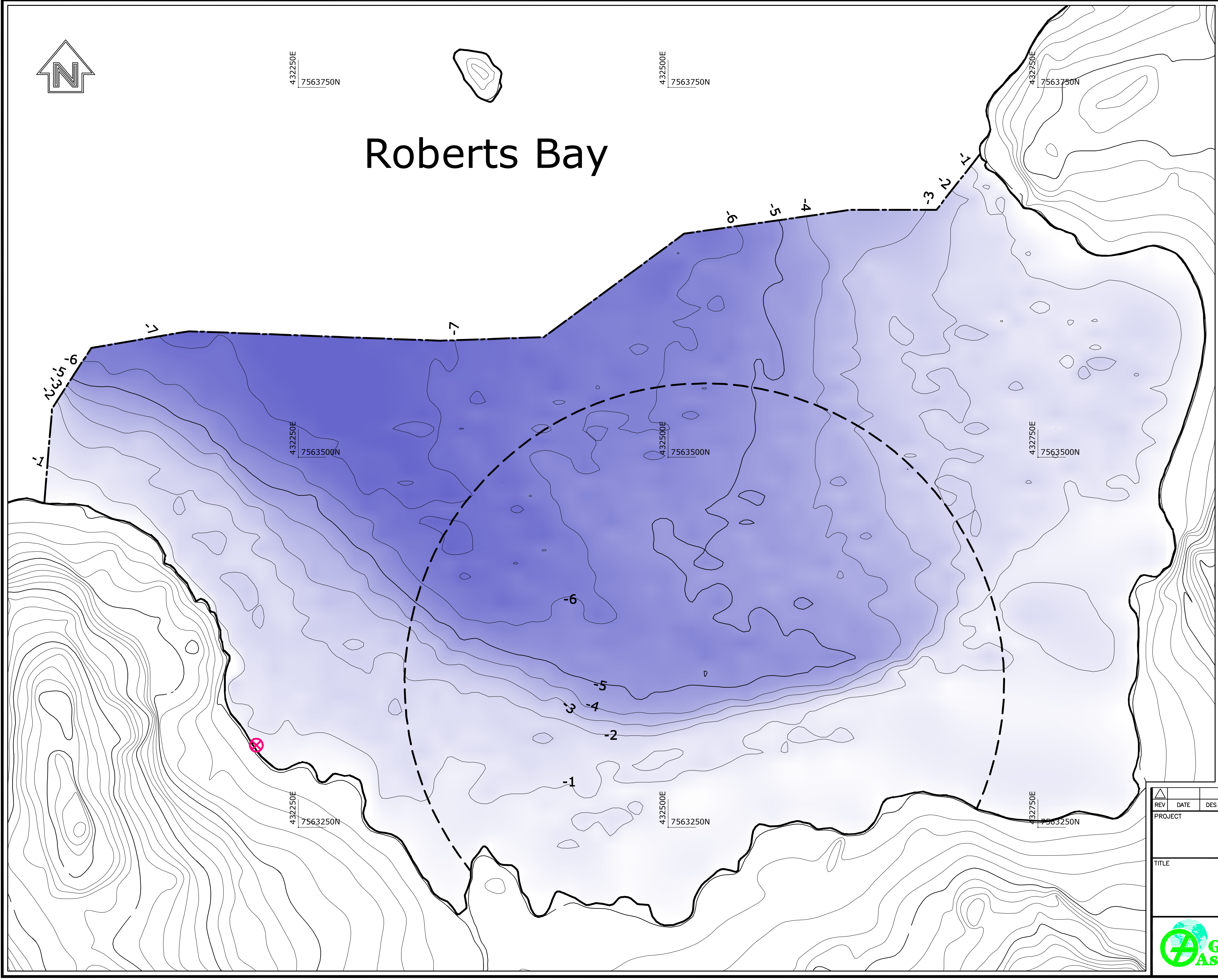
SRK
Hope Bay, NT

TITLE

SITE LOCATION PLAN

Golder Associates

PROJECT No.	06-1419-007		FILE No.	061419007_location
DESIGN			SCALE	NTS REV. 0
CADD	NFT	20061019		
CHECK	JW	20061019	Figure 1	
REVIEW	MM	20061019		



Reference:

Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

Legend:

─	Sea-bed Contour, Major
─	Sea-bed Contour, Minor
━	Shoreline
– – –	SRK Survey Area
—·—	Golder Survey Limit
⊗	Survey Stake

Note:

1. Sea-bed contours are at geodetic elevation are shown at 1m intervals.
2. Grid coordinates are NAD83, Zone 13N.
3. Topographic contour intervals are 1m.
4. Roberts Bay shoreline is shown at -0.05m elevation in topographic base map.
5. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT

SRK
Hope Bay, NT

TITLE

ROBERTS BAY, AREA A
SEA-BED ELEVATIONS

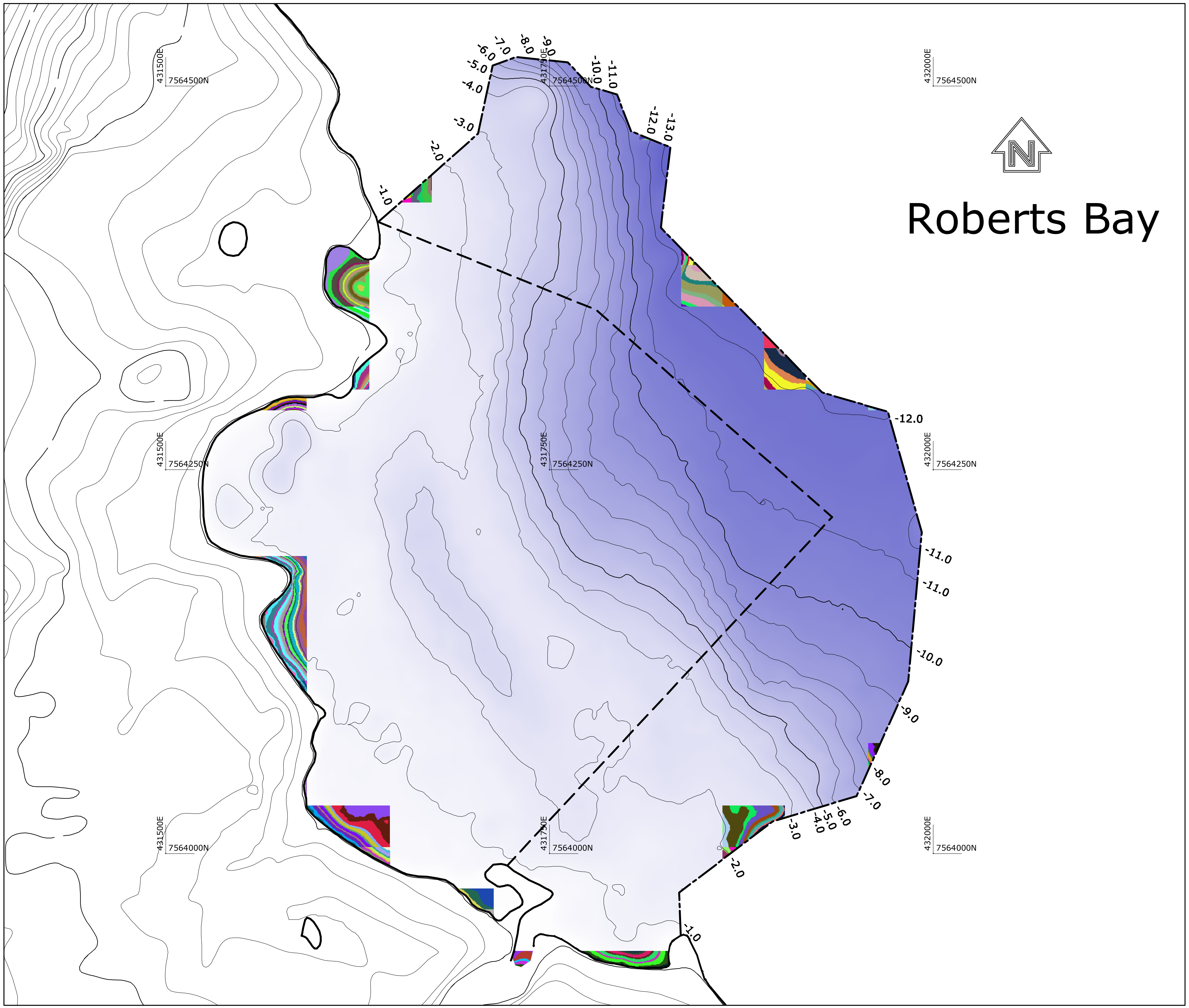
Golder Associates

PROJECT No. 06-1419-007		
DESIGN		
CADD	NFT	20061004
CHECK	JW	20061004
REVIEW	MM	20061004

FILE No. 061419007_bathy_rob-a
SCALE As Shown
REV. 0

Figure 2

Drawing: O:\Active_2006\1419\06-1419-007 Hope Bay Bathymetry SRK\9-Cad\061419007_bathy_rob-a.dwg Plot: 2006/10/20, 14:38 By: ntaylor



Reference:

Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

Legend:

─	Sea-bed Contour, Major
─	Sea-bed Contour, Minor
━	Shoreline
– – –	SRK Survey Area
—·—	Golder Survey Limit
⊗	Survey Stake

Note:

1. Sea-bed contours at geodetic elevation are shown at 1m intervals.
2. Grid coordinates are NAD83, Zone 13N.
3. Topographic contour intervals are 1m.
4. Roberts Bay shoreline is shown at -0.05m elevation in topographic base map.
5. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
△						

PROJECT

SRK
Hope Bay, NT

TITLE

ROBERTS BAY, AREA B
SEA-BED ELEVATIONS

Golder Associates

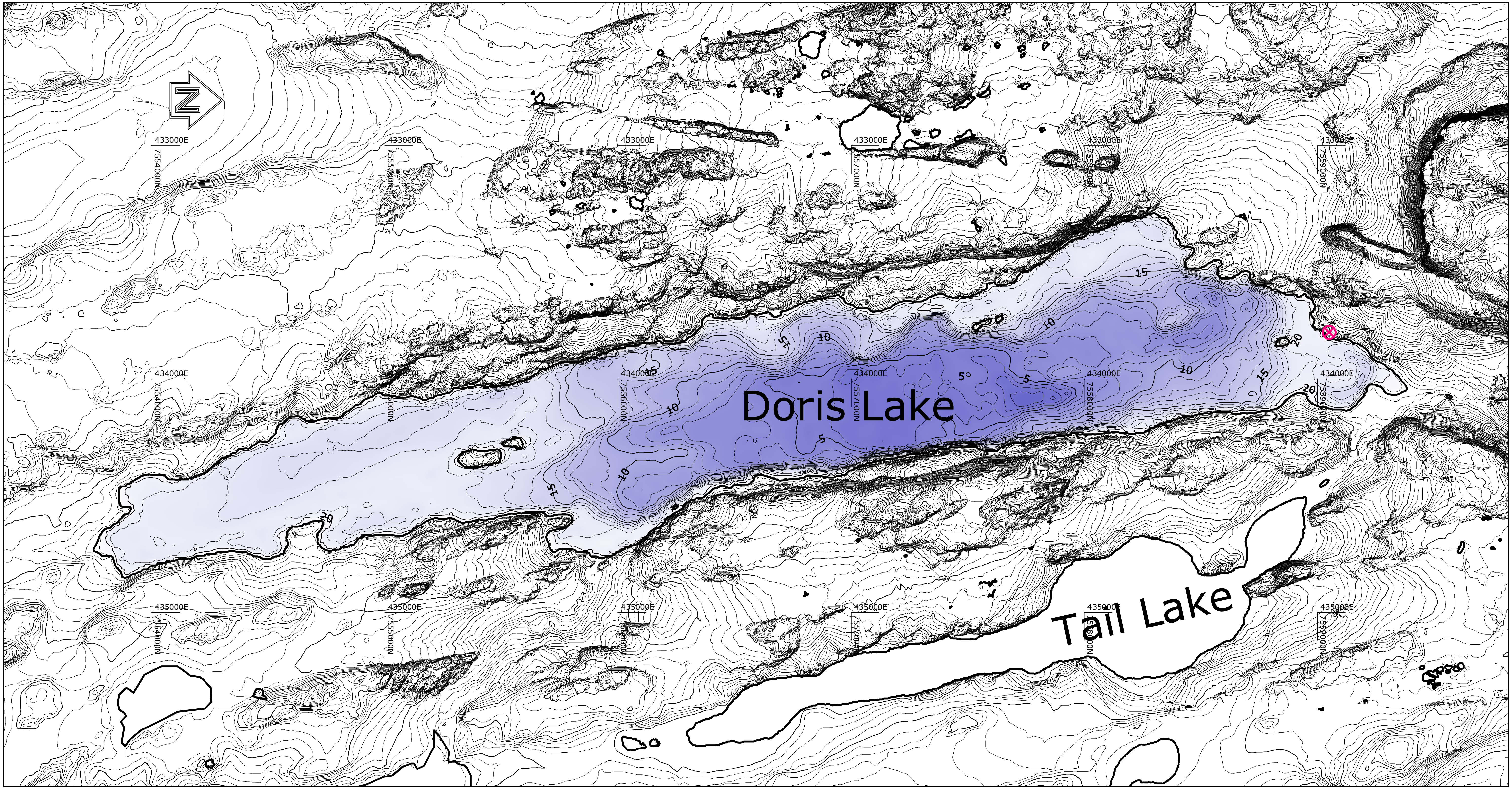
PROJECT No. 06-1419-007		
DESIGN		
CADD	NFT	20061004
CHECK	JW	20061004
REVIEW	MM	20061004

FILE No. 061419007_bathy_rob-b
SCALE As Shown
REV. 0

Figure 3

Drawing: O:\Active_2006\1419\06-1419-007 Hope Bay Bathymetry SRK\9-Cad\061419007_bathy_rob-b.dwg Plot: 2006/10/20, 14:37 By: ntaylor

Drawing: O:\Active_2006\1419\06-1419-007 Hope Bay Bathymetry SRK\9-Cad\061419007_bathy_doris.dwg Plot: 2006/10/20, 14:36 By: ntaylor



Legend:

- Lake-bed Contour, Major
- Lake-bed Contour, Minor
- Shoreline
- ⊗ Survey Stake

Note:

1. Lake-bed contours at geodetic elevation are shown at 1m intervals.
2. Grid coordinates are NAD83, Zone 13N.
3. Topographic contour intervals are 1m and 2m.
4. Doris Lake shoreline at +21.42m elevation geodetic interpolated from topography and survey data.
5. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

Reference:

Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

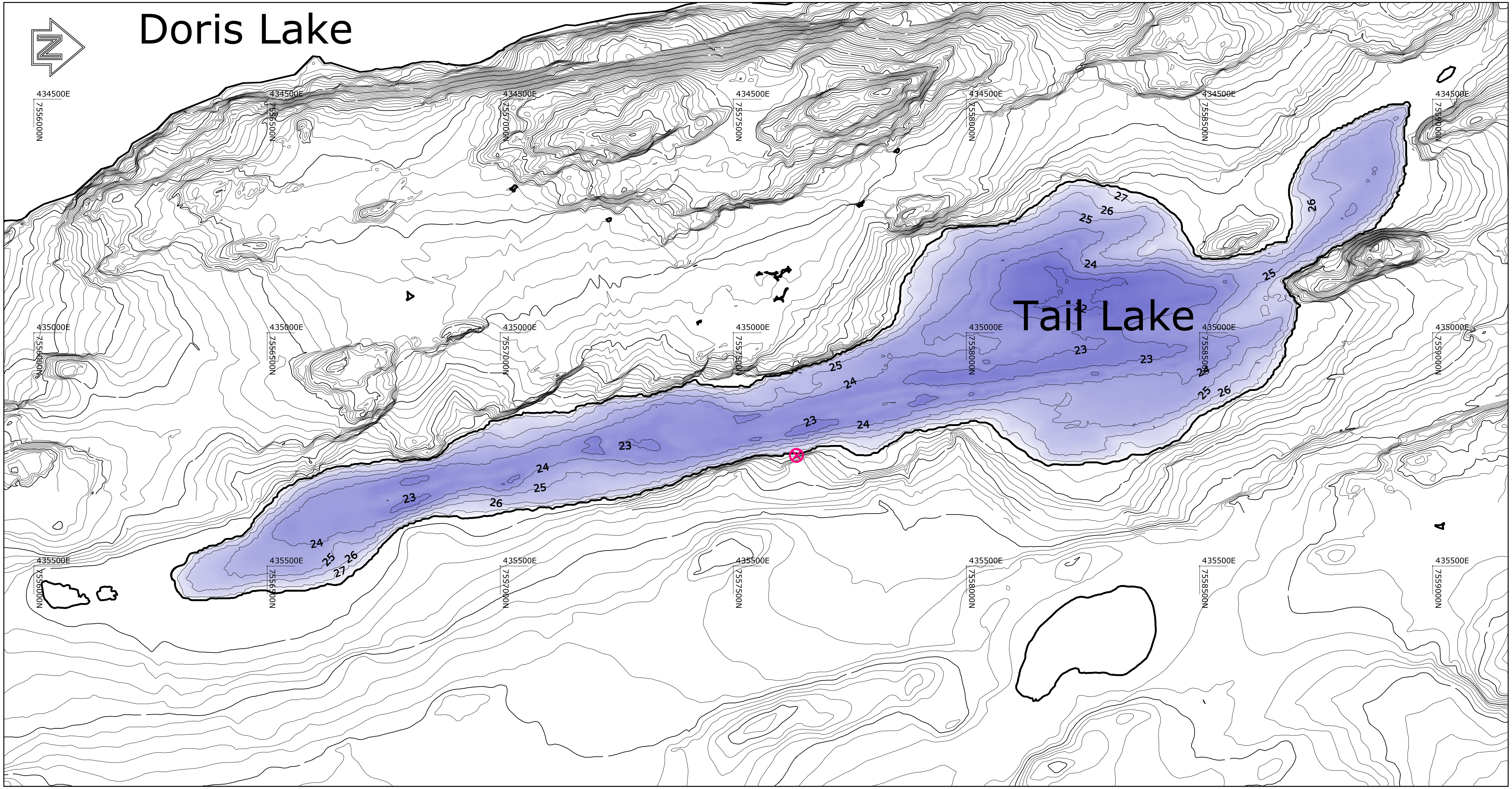
REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
SRK
Hope Bay, NT

TITLE
DORIS LAKE
LAKE-BED ELEVATIONS

Golder Associates

PROJECT No. 06-1419-007			FILE No. 061419007_bathy_doris	
DESIGN			SCALE As Shown	REV. 0
CADD	NFT	20061005	Figure 4	
CHECK	JW	20061005		
REVIEW	MM	20061005		



Legend:

- Lake-bed Contour, Major
- Lake-bed Contour, Minor
- Shoreline
- ⊗ Survey Stake

Note:

1. Lake-bed contours at geodetic elevation are shown at 1m intervals.
2. Grid coordinates are NAD83, Zone 13N.
3. Topographic contour intervals are 1m and 2m.
4. Tail Lake shoreline at +28.12m elevation geodetic interpolated from topography and survey data.
5. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

Reference:

Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

△						
REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT

SRK
Hope Bay, NT

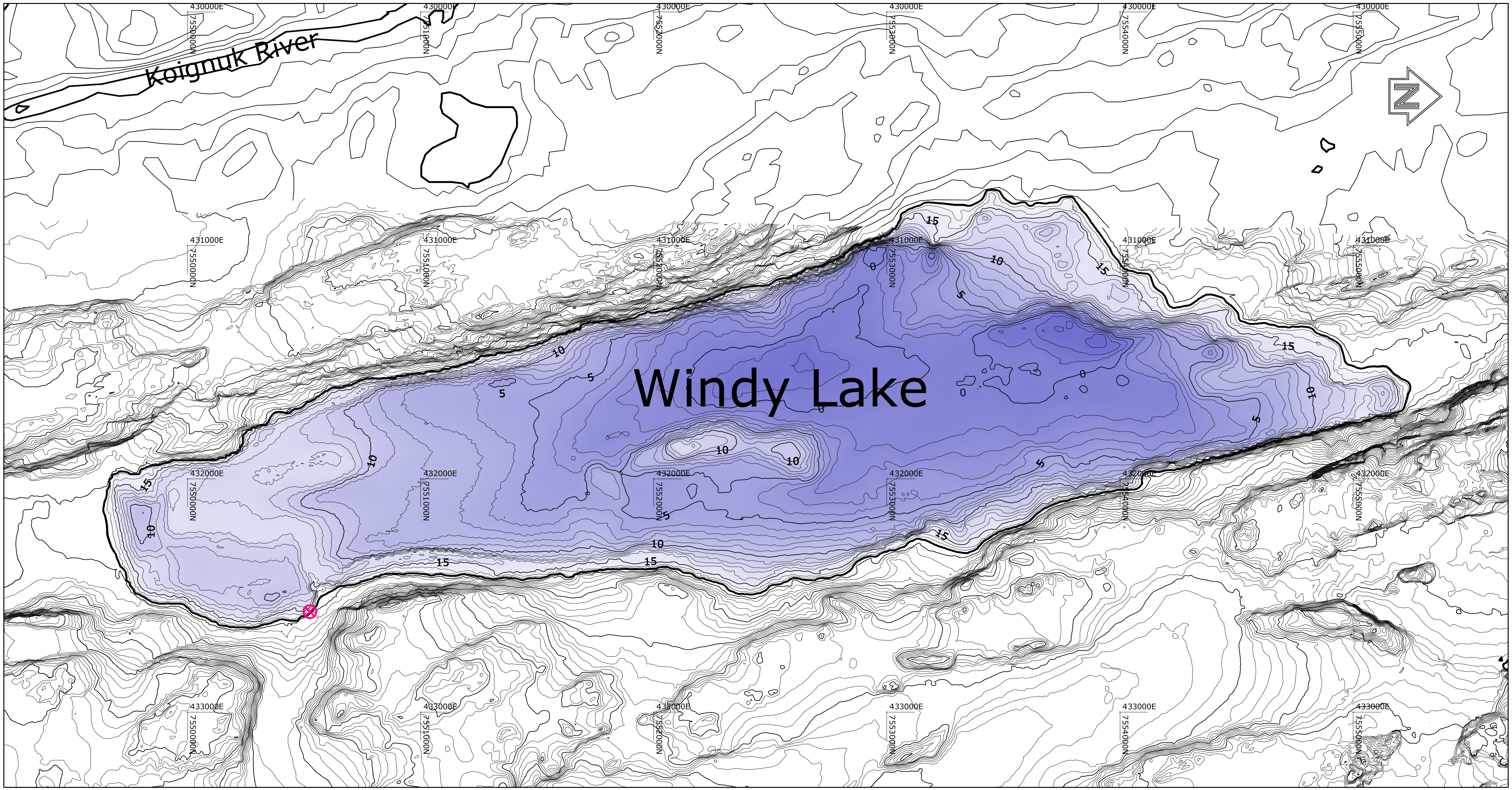
TITLE

TAIL LAKE
LAKE-BED ELEVATIONS

Golder Associates

PROJECT No.	06-1419-007		FILE No.	061419007_bathy_tail
DESIGN			SCALE	As Shown REV. 0
CADD	NFT	20061004		
CHECK	JW	20061004	Figure 5	
REVIEW	MM	20061004		

Drawing: O:\Active_2006\1419\06-1419-007 Hope Bay Bathymetry SRK\9-Cad\061419007_bathy_tail.dwg Plot: 2006/10/20, 14:36 By: ntaylor



Legend:

———	Lake-bed Contour, Major
———	Lake-bed Contour, Minor
━━━	Shoreline
⊗	Survey Stake

Note:

1. Lake-bed contours at geodetic elevation are shown at 1m intervals.
2. Grid coordinates are NAD83, Zone 13N.
3. Topographic contour intervals are 2m, except for coarse topography to west of lake at 10m intervals.
4. Windy Lake shoreline at +18.235m elevation geodetic interpolated from topography and survey data.
5. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

Reference:

Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT

SRK
Hope Bay, NT

TITLE

WINDY LAKE
LAKE-BED ELEVATIONS

Golder Associates

PROJECT No. 06-1419-007			FILE No. 061419007_bathy_windy	
DESIGN			SCALE As Shown	REV. 0
CADD	NFT	20061005	Figure 6	
CHECK	JW	20061005		
REVIEW	MM	20061005		

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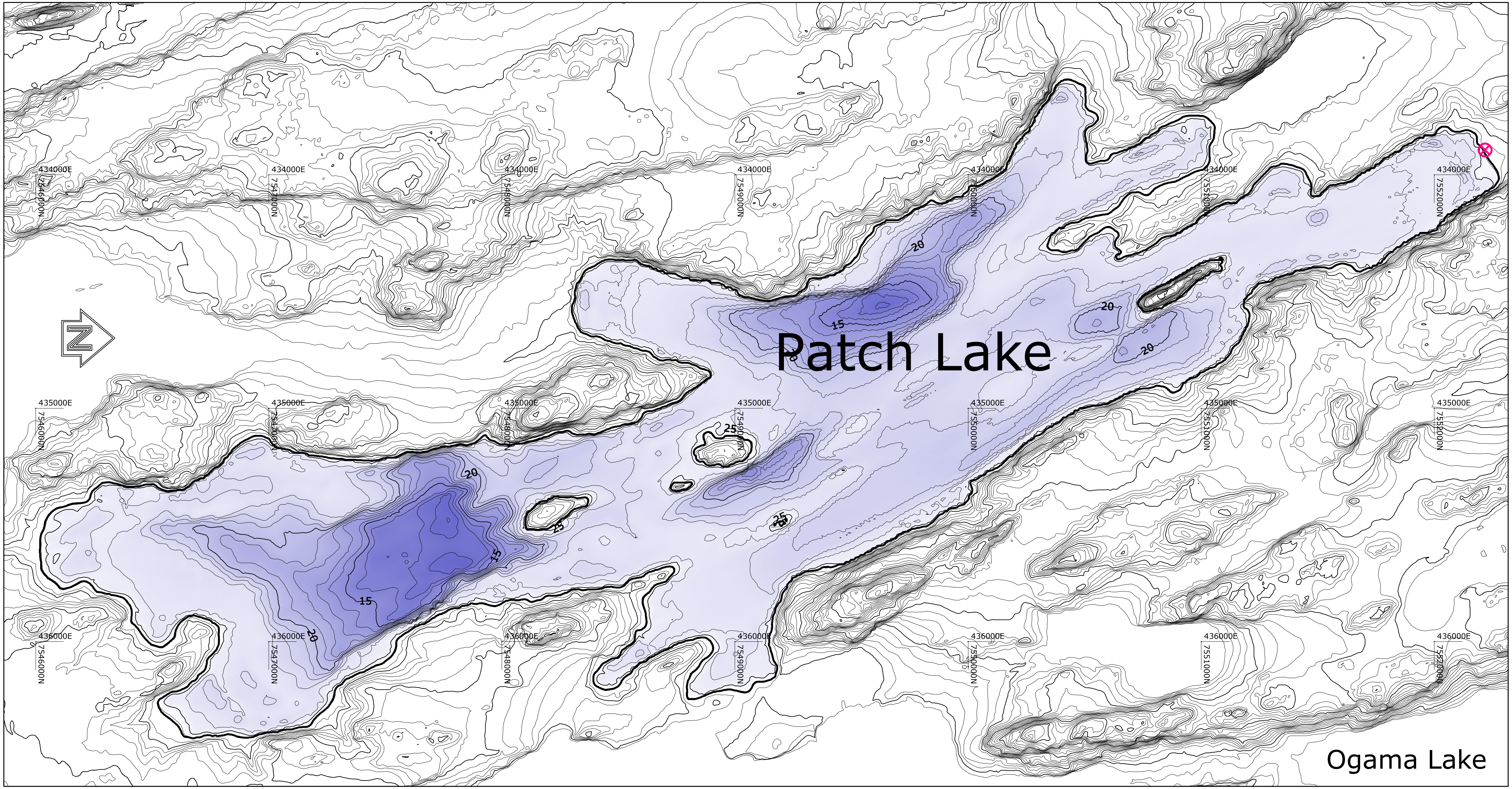

Legend:

- Lake-bed Contour, Major
- Lake-bed Contour, Minor
- Shoreline
- ⊗ Survey Stake

Note:

1. Lake-bed contours at geodetic elevation are shown at 1m intervals.
2. Grid coordinates are NAD83, Zone 13N.
3. Topographic contour intervals are 2m.
4. Patch Lake shoreline at +26.275m geodetic elevation interpolated from topography and survey data.
5. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

Reference:

Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
△						

PROJECT

SRK
Hope Bay, NT

TITLE

PATCH LAKE
LAKE-BED ELEVATIONS

Golder Associates

PROJECT No. 06-1419-007			FILE No. 061419007_bathy_patch	
DESIGN			SCALE As Shown	REV. 0
CADD	NFT	20061003	Figure 7	
CHECK	JW	20061004		
REVIEW	MM	20061004		

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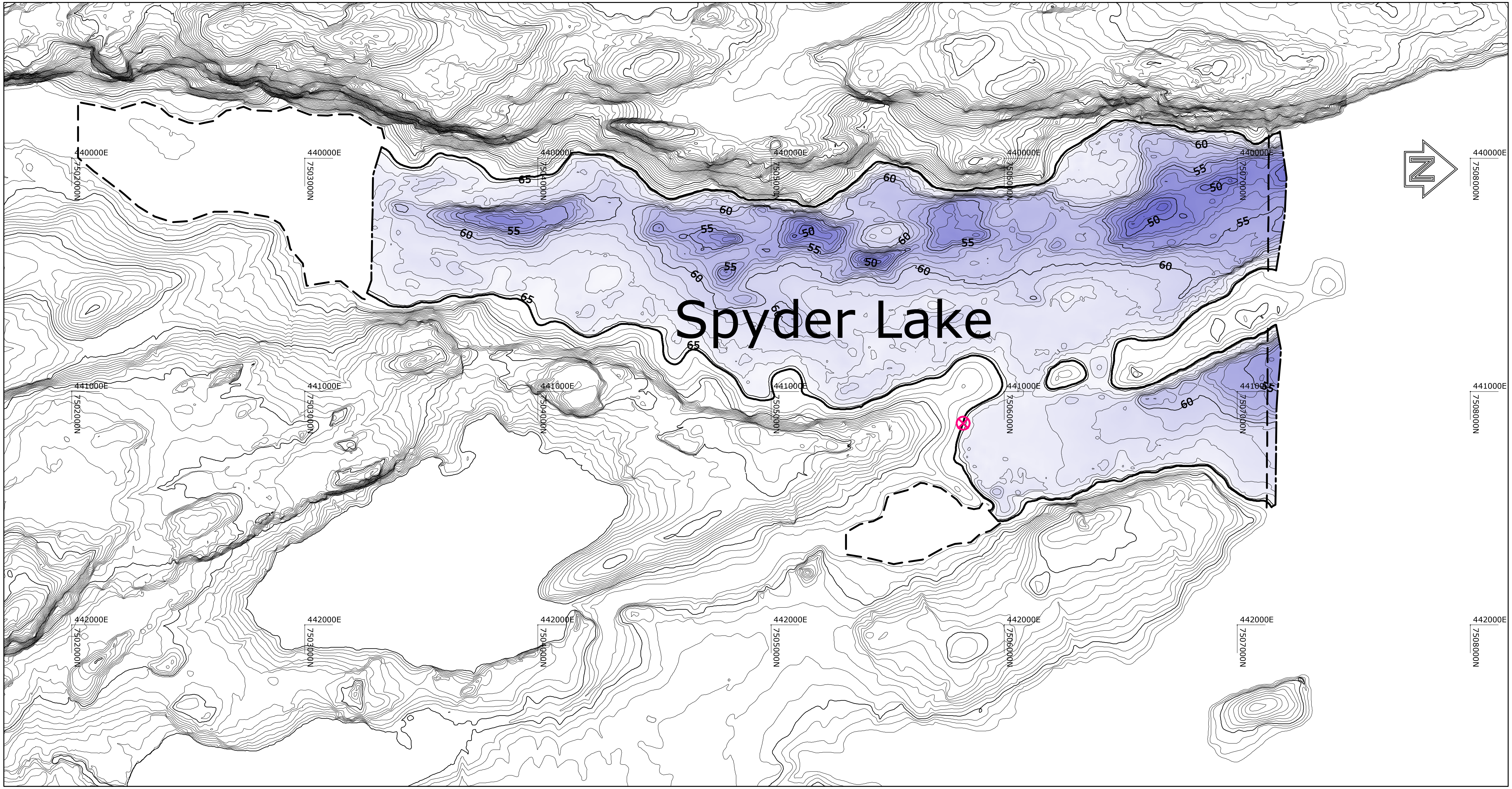


Legend:

Lake-bed Contour, Major
Lake-bed Contour, Minor
Shoreline
SRK Survey Area
Golder Survey Limit
⊗ Survey Stake

Note:

1. Lake-bed contours at geodetic elevation are shown at 1m intervals.
2. Grid coordinates are NAD83, Zone 13N.
3. Topographic contour intervals are 1m.
4. Spyder Lake shoreline at +65.625 geodetic elevation interpolated from topography and survey data.
5. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

Reference:

Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW

PROJECT
SRK
Hope Bay, NT

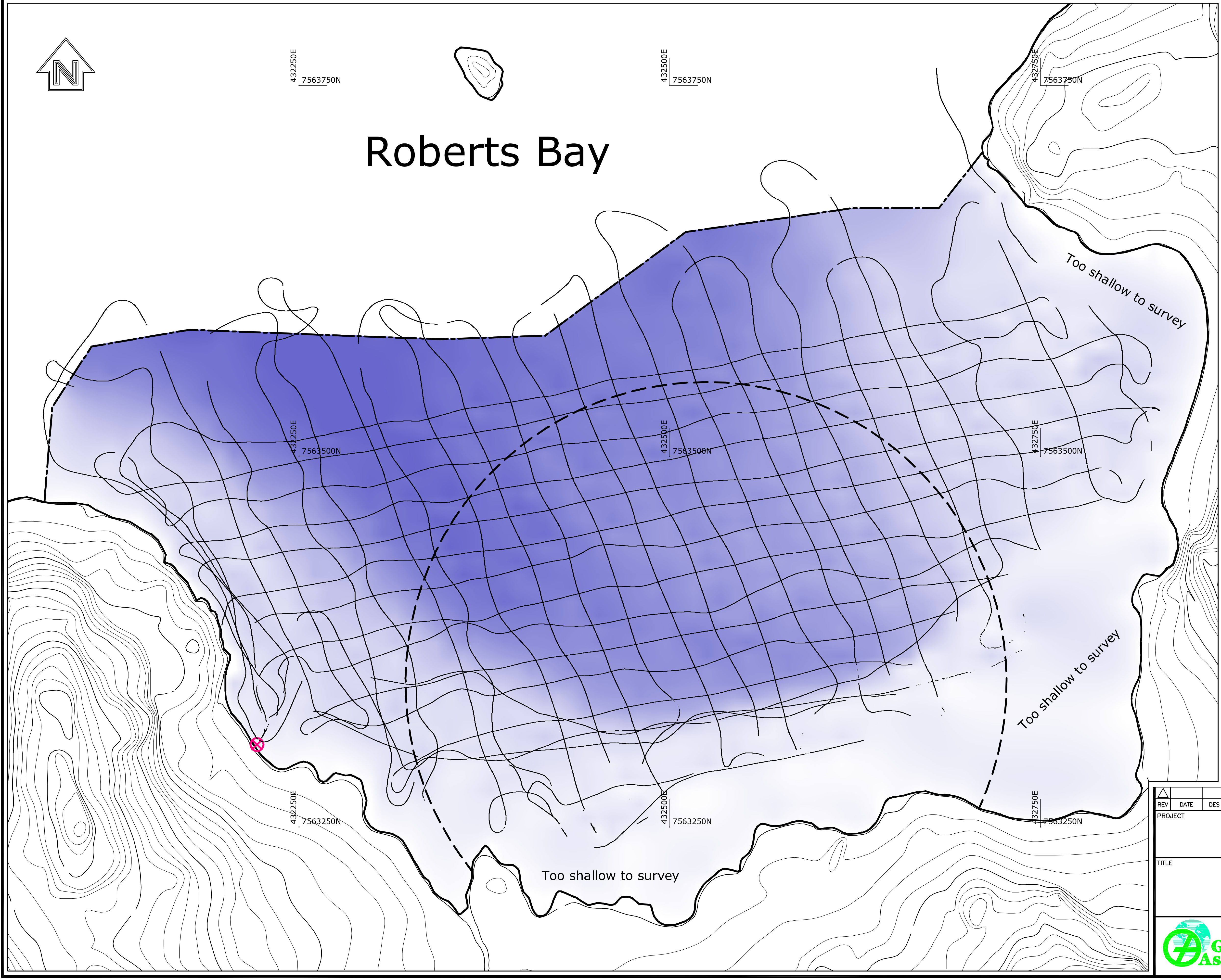
TITLE
SPYDER LAKE
LAKE-BED ELEVATIONS

Golder Associates

PROJECT No. 06–1419–007		
DESIGN		
CADD	NFT	20061004
CHECK	JW	20061004
REVIEW	MM	20061004

FILE No. 061419007_bathy_spyder
SCALE As Shown   REV. 0

Figure 8



Reference:

Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

Legend:

——— Shoreline

— — — SRK Survey Area

—·— Golder Survey Limit

⊗ Survey Stake

Note:

1. Grid coordinates are NAD83, Zone 13N.
2. Topographic contour intervals are 1m.
3. Roberts Bay shoreline is shown at -0.05m elevation in topographic base map.
4. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
△						

PROJECT

SRK
Hope Bay, NT

TITLE

ROBERTS BAY, AREA A
SURVEY TRACKLINES

Golder Associates

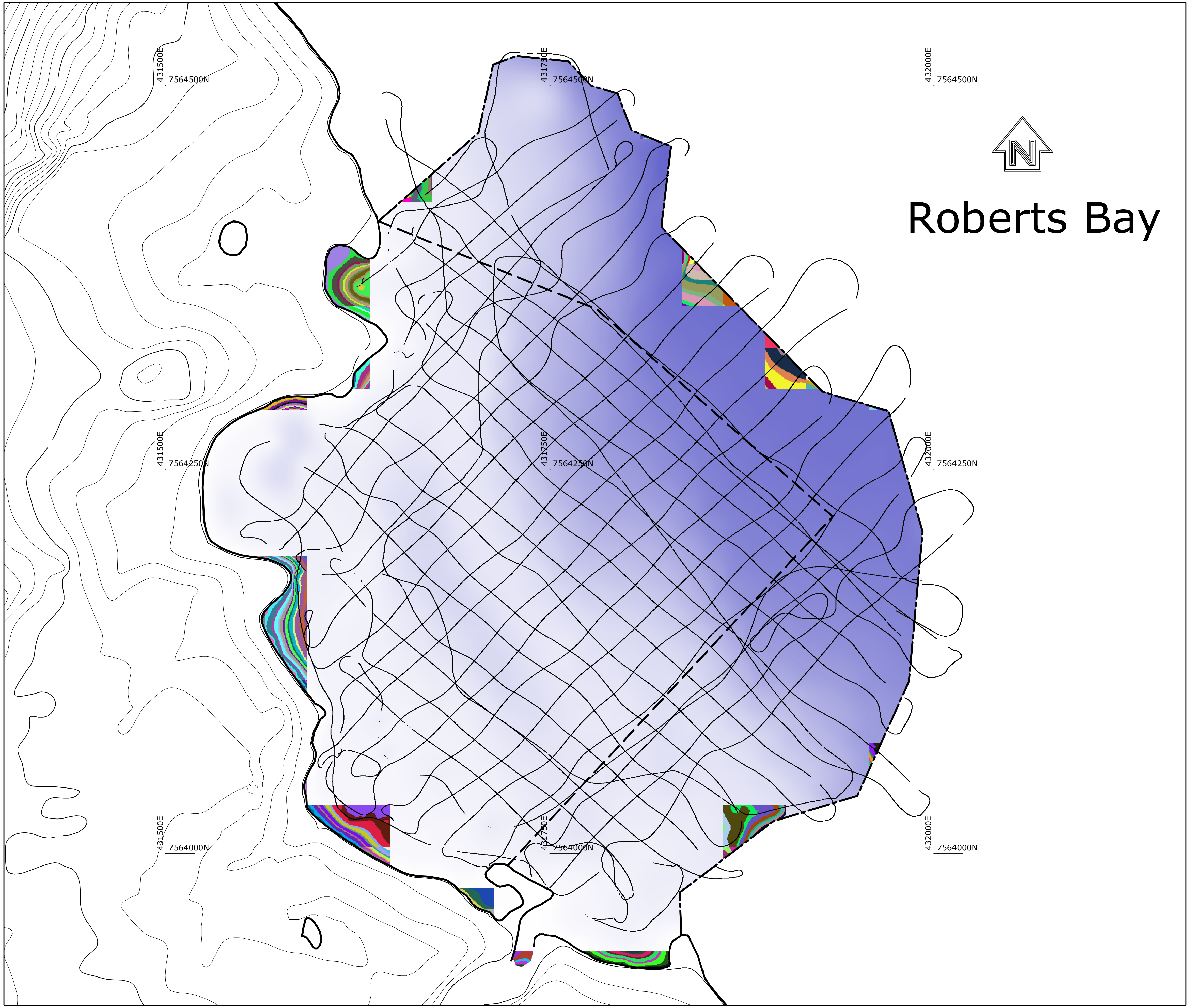
PROJECT No. 06-1419-007		
DESIGN		
CADD	NFT	20061016
CHECK	JW	20061016
REVIEW	MM	20061016

FILE No. 061419007_track_rob-a

SCALE As Shown REV. 0

Figure 9

Drawing: O:\Active_2006\1419\06-1419-007 Hope Bay Bathymetry SRK\9-Cad\061419007_track_rob-a.dwg Plot: 2006/10/20, 14:30 By: ntaylor



Reference:

Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

Legend:

———	Shoreline
— — —	SRK Survey Area
—·—	Golder Survey Limit
⊗	Survey Stake

Note:

1. Grid coordinates are NAD83, Zone 13N.
2. Topographic contour intervals are 1m.
3. Roberts Bay shoreline is shown at -0.05m elevation in topographic base map.
4. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
△						

PROJECT

SRK
Hope Bay, NT

TITLE

ROBERTS BAY, AREA B
SURVEY TRACKLINES

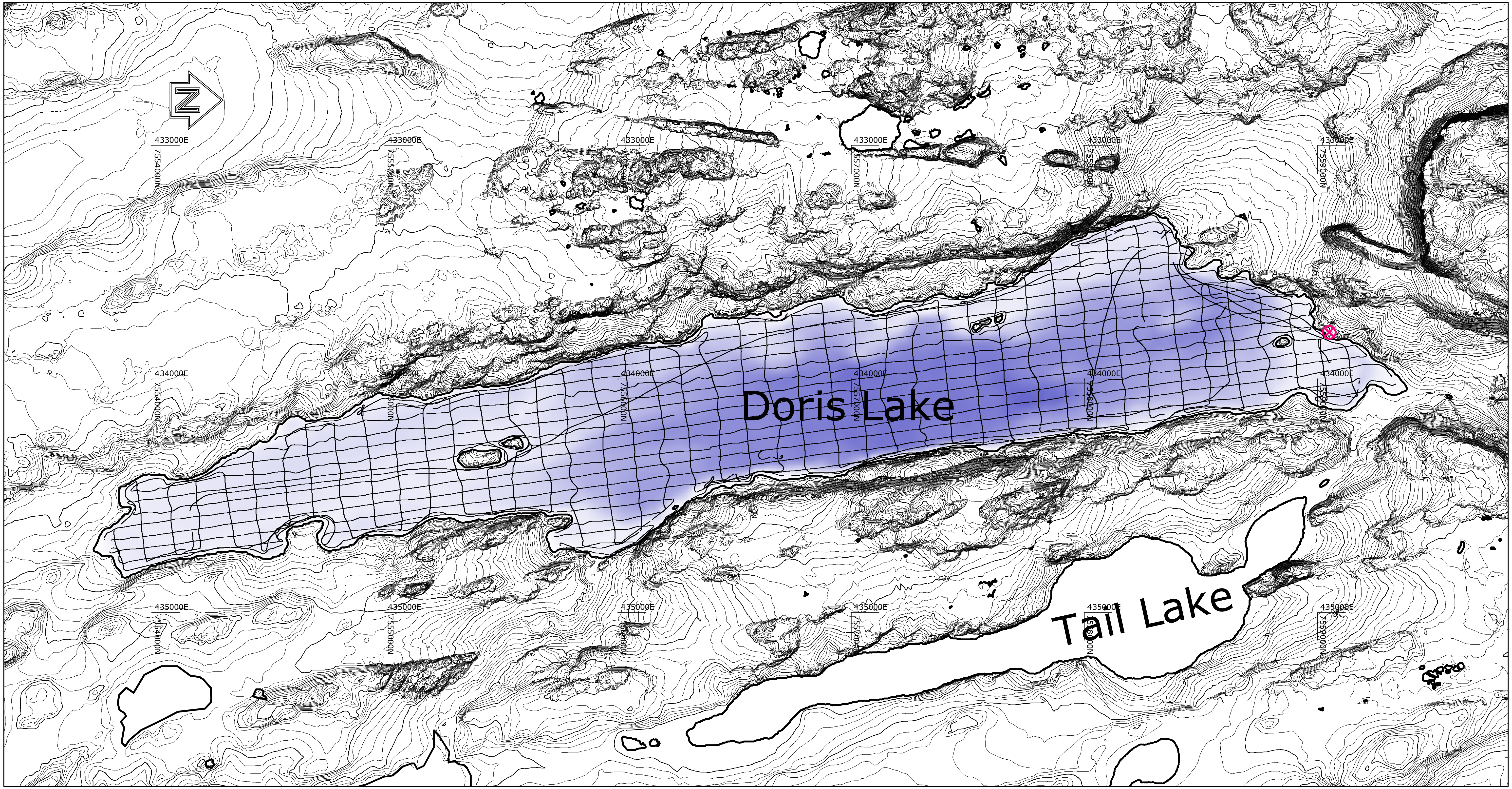
PROJECT No.	06-1419-007	
DESIGN		
CADD	NFT	20061016
CHECK	JW	20061016
REVIEW	MM	20061016

FILE No. 061419007_track_rob-b

SCALE As Shown REV. 0

Figure 10

Drawing: O:\Active_2006\1419\06-1419-007 Hope Bay Bathymetry SRK\9-Cad\061419007_track_rob-b.dwg Plot: 2006/10/20, 14:29 By: ntaylor



Legend:

—	Shoreline
⊗	Survey Stake

Note:

1. Grid coordinates are NAD83, Zone 13N.
2. Topographic contour intervals are 1m and 2m.
3. Doris Lake shoreline at +21.42m elevation geodetic interpolated from topography and survey data.
4. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

Reference:
Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
△						

PROJECT
SRK
Hope Bay, NT

TITLE
DORIS LAKE
SURVEY TRACKLINES

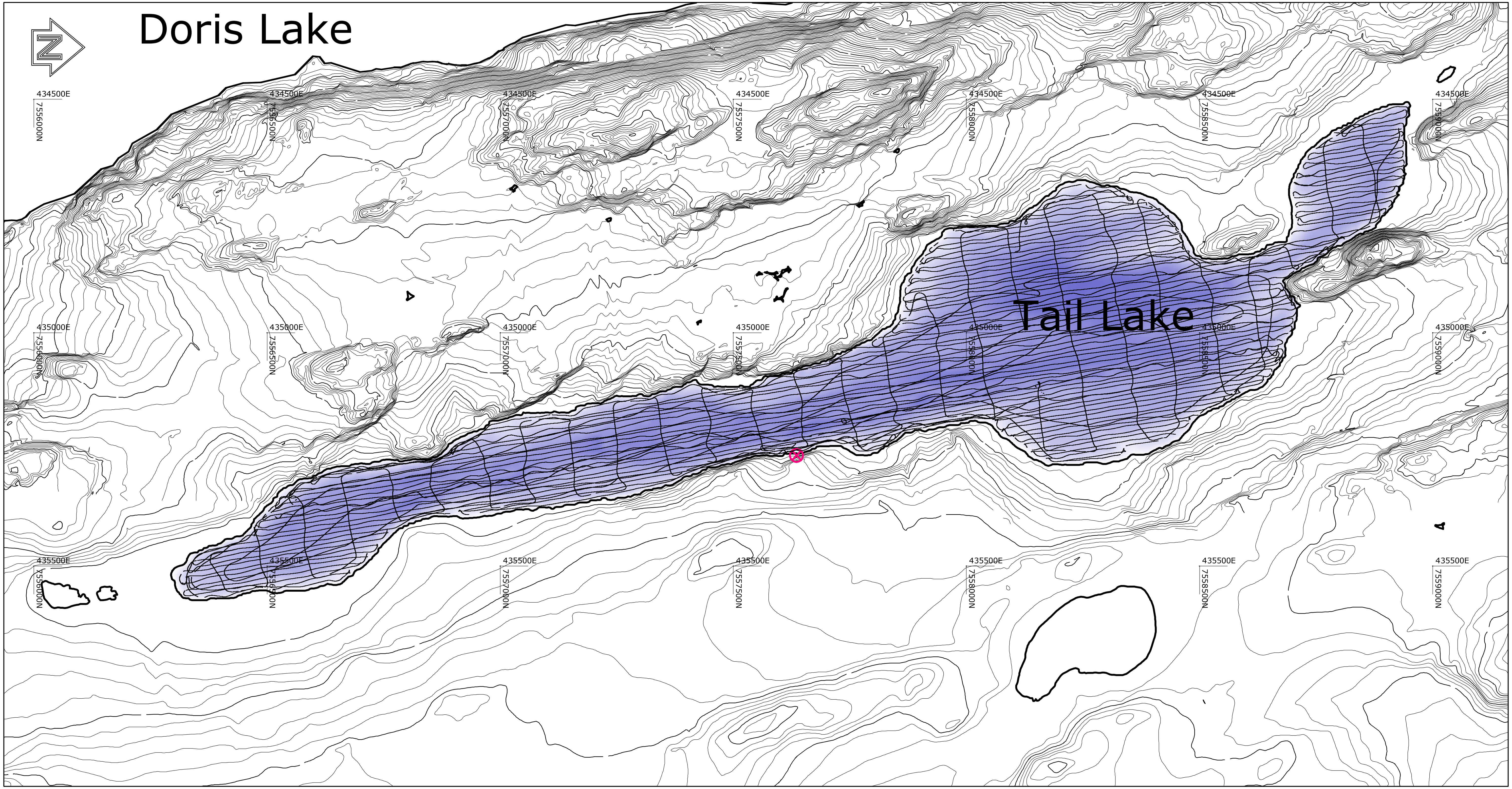
Golder Associates

PROJECT No. 06-1419-007		
DESIGN		
CADD	NFT	20061016
CHECK	JW	20061016
REVIEW	MM	20061016

FILE No. 061419007_track_doris
SCALE As Shown
REV. 0

Figure 11

Drawing: O:\Active_2006\1419\06-1419-007 Hope Bay Bathymetry SRK\9-Cad\061419007_track_doris.dwg Plot: 2006/10/20, 14:28 By: ntaylor



Legend:

- Shoreline
- ⊗ Survey Stake

Note:

1. Grid coordinates are NAD83, Zone 13N.
2. Topographic contour intervals are 1m and 2m.
3. Tail Lake shoreline at +28.12m elevation geodetic interpolated from topography and survey data.
4. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

Reference:

Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
△						

PROJECT

SRK
Hope Bay, NT

TITLE

TAIL LAKE
SURVEY TRACKLINES

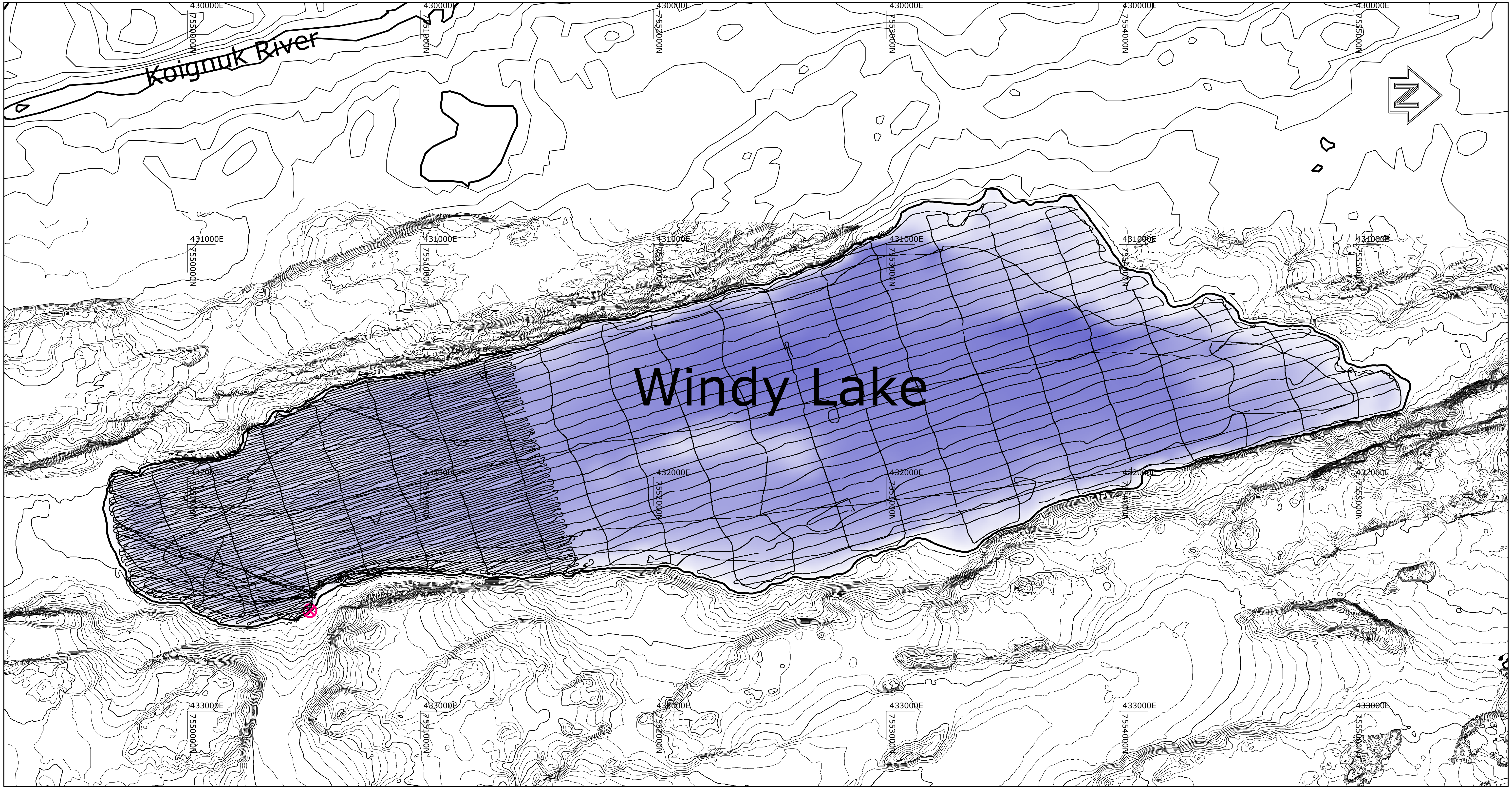
Golder Associates

PROJECT No. 06-1419-007		
DESIGN		
CADD	NFT	20061016
CHECK	JW	20061016
REVIEW	MM	20061016

FILE No. 061419007_track_tail
SCALE As Shown
REV. 0

Figure 12

Drawing: O:\Active_2006\1419\06-1419-007 Hope Bay Bathymetry SRK\9-Cad\061419007_track_tail.dwg Plot: 2006/10/20, 14:26 By: ntaylor



Drawing: O:\Active_2006\1419\06-1419-007 Hope Bay Bathymetry SRK\9-Cad\061419007_track-windy.dwg Plot: 2006/10/20, 14:23 By: ntaylor

Legend:

▬	Shoreline
⊗	Survey Stake

Note:

1. Grid coordinates are NAD83, Zone 13N.
2. Topographic contour intervals are 2m, except for coarse topography to west of lake at 10m intervals.
3. Windy Lake shoreline at +18.235m elevation geodetic interpolated from topography and survey data.
4. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

Reference:

Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
△						

PROJECT

SRK
Hope Bay, NT

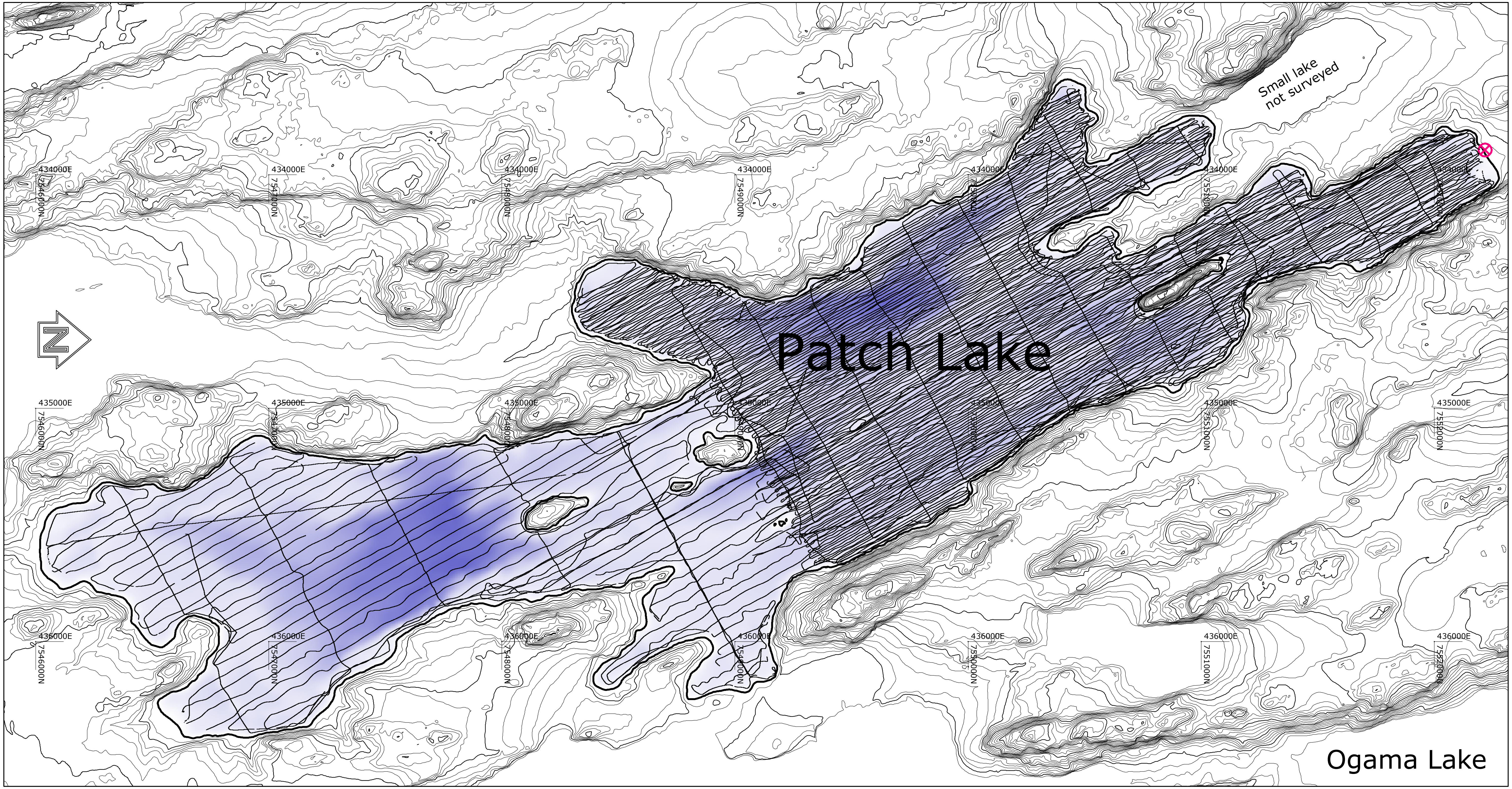
TITLE

WINDY LAKE
SURVEY TRACKLINES

Golder Associates

PROJECT No.	06-1419-007		FILE No.	061419007_track_windy
DESIGN			SCALE As Shown	REV. 0
CADD	NFT	20061016		
CHECK	JW	20061016	Figure 13	
REVIEW	MM	20061016		

Drawing: O:\Active_2006\1419\06-1419-007 Hope Bay Bathymetry SRK\9-Cad\061419007_track-patch.dwg   Plot: 2006/10/20, 14:17   By: ntaylor



Legend:

▬	Shoreline
⊗	Survey Stake

Note:

1. Grid coordinates are NAD83, Zone 13N.
2. Topographic contour intervals are 2m.
3. Patch Lake shoreline at +26.275m geodetic elevation interpolated from topography and survey data.
4. Small lake at NW end of Patch Lake was not surveyed due to cliff blocking GPS signal.
5. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

Reference:

Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

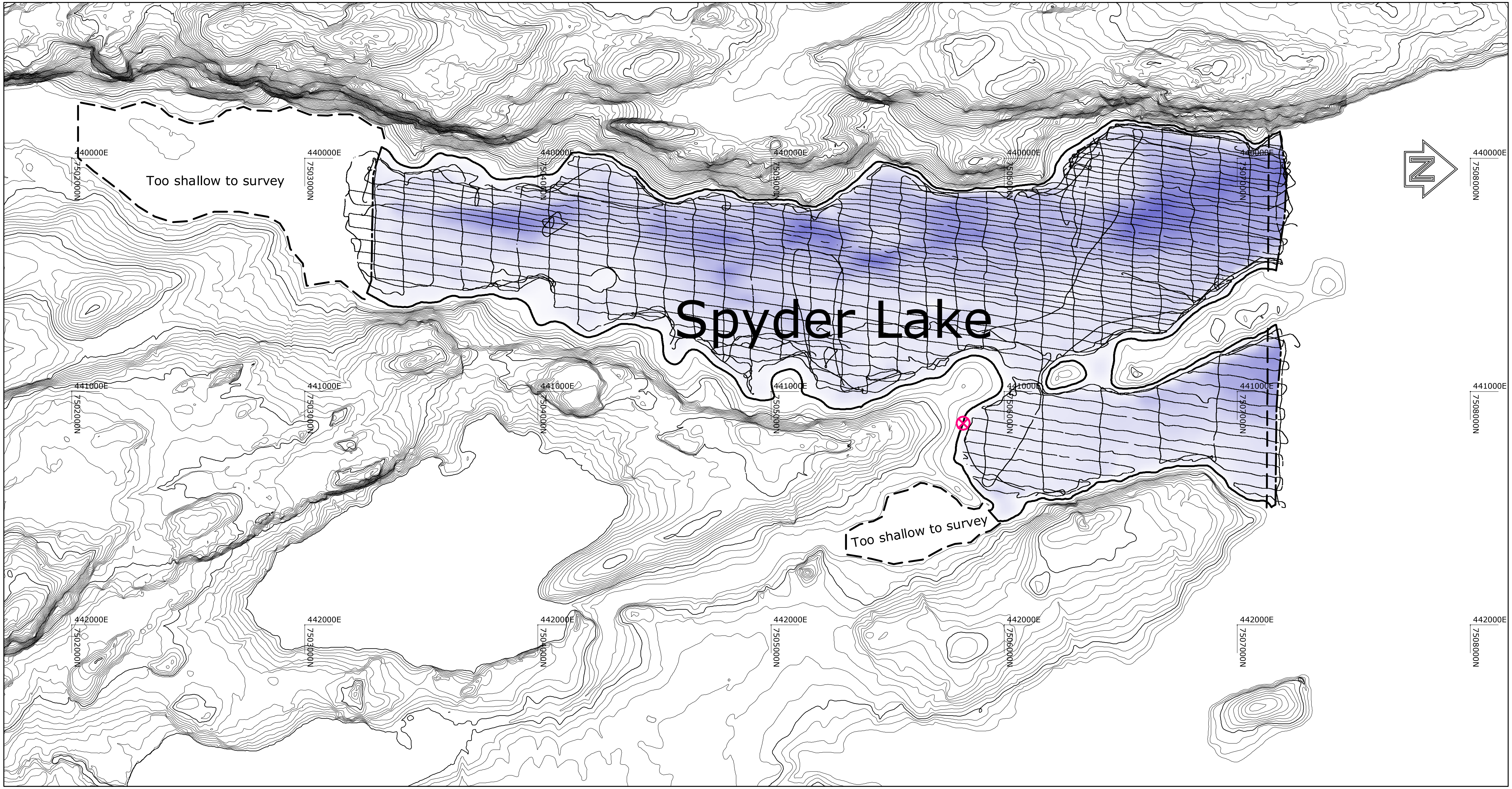
REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
△						

PROJECT
SRK
Hope Bay, NT

TITLE
PATCH LAKE
SURVEY TRACKLINES

Golder Associates

PROJECT No.	06-1419-007		FILE No.	061419007_track_patch
DESIGN			SCALE As Shown	REV. 0
CADD	NFT	20061016	Figure 14	
CHECK	JW	20061016		
REVIEW	MM	20061016		



Legend:

——— Shoreline

— — — SRK Survey Area

—·— Golder Survey Limit

⊗ Survey Stake

Note:

1. Grid coordinates are NAD83, Zone 13N.
2. Topographic contour intervals are 1m.
3. Spyder Lake shoreline at +65.625 geodetic elevation interpolated from topography and survey data.
4. Figure to be read in conjunction with Golder report "Rpt1019_06 - SRK - Hope Bay Bathymetry".

Reference:

Topographic information (NAD83, Zone 13N) generated by BHP 1997 and provided by MHBL.

REV	DATE	DES	REVISION DESCRIPTION	CADD	CHK	RVW
△						

PROJECT
SRK
Hope Bay, NT

TITLE
SPYDER LAKE
SURVEY TRACKLINES

PROJECT No.	06-1419-007		FILE No.	061419007_track_spyder
DESIGN			SCALE As Shown	REV. 0
CADD	NFT	20061016	Figure 15	
CHECK	JW	20061016		
REVIEW	MM	20061016		

Golder Associates

Drawing: O:\Active_2006\1419\06-1419-007 Hope Bay Bathymetry SRK\9-Cad\061419007_track-spyder.dwg Plot: 2006/10/20, 14:14 By: ntaylor

Appendix B

Phase III Foundation Investigation (SRK 2006a)

Phase III Foundation Investigation Proposed Roberts Bay Jetty Location, Doris North Project, Nunavut, Canada

Prepared for

Miramar Hope Bay Limited

Prepared by

SRK Consulting
Engineers and Scientists

August 2006

Phase III Foundation Investigation Proposed Roberts Bay Jetty Location, Doris North Project, Nunavut, Canada

Miramar Hope Bay Limited

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SRK Project Number 1CM014.008-260

August 2006

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Reviewed by
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Principal Engineer

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1 Introduction

1.1 Background

Miramar Hope Bay Limited (MHBL) is planning a new gold mine in the Hope Bay Belt in Nunavut. This project, called the Doris North Project, lies on the Artic coastline, and as a result annual re-supply for the mine will be via sealift. A permanent off-loading facility will be constructed at Roberts Bay, which is approximately 4 km north of the proposed mine site, as indicated in Figure 1. This facility will be a 103 m long continuous rockfill jetty (SRK 2005a).

In preparation for final detailed engineering designs of this jetty, MHBL contracted SRK Consulting (Canada) Inc. (SRK) to carry out additional geotechnical investigations to further characterize the foundation conditions. This study, which was carried out in May 2006, was intended to fill the remaining data gaps identified during two previous geotechnical foundation investigations in April 2004 (SRK 2004) and April 2005 (SRK 2005b).

This report presents the results of the study as described. It includes drill logs, core photos as well as the complete laboratory testing data.

1.2 Summary of Drill Program

Seven drill holes were completed at the head of the jetty as illustrated in Figure 2. Drill holes SRK06-11 to SRK06-17 was advanced using a portable vibro-core drill, and in-tact (undisturbed) soil samples were collected and sent for laboratory characterization in order to obtain design parameters to be used in the detailed design of the jetty.

Seasonal weather conditions prevailed for the duration of the drilling operation. Winds were generally from the north and northeast at 5-20 km/h, with daytime temperatures up to 4°C and overnight lows of -10°C. During daylight hours, conditions ranged from sunny to overcast.

2 Methodology

2.1 Drilling

The holes were completed using a portable BQ (size 36.5 mm) vibro-core system, complete with vibration pack and rod extraction using a hydraulic power pack. The drilling equipment was supplied by Rocky Mountain Soil Sampling Inc. (RMSS), from Hornby Island, British Columbia. The drill was operated by a two-man crew from RMSS, working a single 12-hour day shift.

Actual drilling was done from the top of the sea ice in Roberts Bay. Drilling consisted of penetrating hollow drill rods through the marine sediments (foundation soils). The driving force was a combination of gravity and vibration. Continuous (undisturbed) samples were gathered during rod penetration. These samples were subsequently pumped out by a piston using a hydraulic pack after all the rods were extracted at the completion of the hole.

SRK engineer, Mr. Alvin Tong, E.I.T. supervised the drilling and logged and photographed the extracted core on-site as drilling progressed. Selected representative samples were prepared and shipped to EBA Engineering’s soil testing laboratories in Yellowknife and Edmonton for geotechnical characterization. All remaining soil core is stored in core boxes, outside, under ambient conditions at Windy Camp, approximately 11 km south of Roberts Bay.

The drill hole locations were set out by the SRK engineer using a hand-held GPS according to planning co-ordinates selected by SRK. After completion of the drilling, Miramar surveyed in the actual hole locations.

2.2 Laboratory Testing

18 samples was collected from the extracted core and sent to the laboratory for geotechnical testing. Although theoretically all samples were “undisturbed”, the soft nature of the material did result in some consolidation, and handling of the core also resulted in the “undisturbed” nature of the core being affected. In reality the SRK engineer could only reliably classify six of the collected samples are truly “undisturbed”, as depicted in Table 1.

From the 18 samples, SRK selected 13 samples only (including the six “undisturbed” samples) on which to carry out geotechnical testing as listed in Table 2.

Table 1: Samples collected and sent to laboratory

Sample [ID:Depth (Type)][1]	Sample Condition
SRK06-11-01: 1.1m (SP)[2]	Bulk – disturbed
SRK06-12-01: 4.0m (CL)[3]	Bulk – disturbed
SRK06-12-02: 7.9m (CL)	In-tact – undisturbed
SRK06-12-03: 18.6m (CL)	Bulk – disturbed
SRK06-13-01: 5.5m (CL)	In-tact – undisturbed
SRK06-13-02: 18.6m (CL)	Bulk – disturbed
SRK06-14-01: 5.1m (CL)	Bulk – disturbed
SRK06-14-02: 6.9m (CL)	In-tact – undisturbed
SRK06-14-03: 17.7m (SP)	Bulk – disturbed
SRK06-15-01: 3.9m (ML)[4]	In-tact – undisturbed
SRK06-15-02: 7.0m (CL)	Bulk – disturbed
SRK06-15-03: 5.6m (SP)	Bulk – disturbed
SRK06-16-01: 5.5m (CL)	Bulk – disturbed
SRK06-16-02: 8.3m (CL)	In-tact – undisturbed
SRK06-16-03: 17.7m (SP)	Bulk – disturbed
SRK06-17-01: 2.8m (SP)	Bulk – disturbed
SRK06-17-02: 8.0m (CL)	Bulk – disturbed
SRK06-17-03: 12.8m (CL)	In-tact – undisturbed

1. Soil type is designated soil symbol according to the Unified Soil Classification System (USCS).
2. SP = Poorly graded sand.
3. CL = Clay.
4. ML = Silt.

Table 2: Laboratory testing program

Sample [ID:Depth (Type)][1]	Natural Moisture Content	Particle Size Distribution		Atterberg Limits	Salinity	Triaxial (UU)[5]	Consolidation
		Sieve	Hydrometer				
SRK06-11-01: 1.1m (SP)[2]	✓	✓	✓	✓	✓		
SRK06-12-02: 7.9m (CL)[3]	✓	✓	✓	✓		✓	
SRK06-12-03: 18.6m (CL)	✓	✓	✓	✓			
SRK06-13-01: 5.5m (CL)	✓	✓	✓	✓			
SRK06-13-02: 18.6m (CL)	✓	✓	✓	✓			
SRK06-14-02: 6.9m (CL)	✓	✓	✓	✓			✓
SRK06-15-01: 3.9m (ML)[4]	✓	✓	✓	✓			
SRK06-15-02: 7.0m (CL)	✓	✓	✓	✓			
SRK06-16-01: 5.5m (CL)	✓	✓	✓	✓			
SRK06-16-02: 8.3m (CL)	✓	✓	✓	✓		✓	
SRK06-17-01: 2.8m (SP)	✓	✓	✓	✓			
SRK06-17-02: 8.0m (CL)	✓	✓	✓	✓			
SRK06-17-03: 12.8m (CL)	✓	✓	✓	✓			✓

1. Soil type is designated soil symbol according to the Unified Soil Classification System (USCS).
2. SP = Poorly graded sand.
3. CL = Clay.
4. ML = Silt.
5. UU = Unconsolidated Undrained triaxial shear test.

3 Results

3.1 Drilling Hole Locations

The as-built collar locations differed slightly from those set out using a hand-held GPS; however, Table 3 list the actual survey co-ordinates as provided by MHBL after completion of the holes.

Table 3: As-built drill hole coordinates

Hole ID	Northing[1]	Easting[1]	Elevation[2]	Inclination
SRK06-11	7563305.4	432543.8	-0.2	-90°
SRK06-12	7563336.7	432549.8	-0.5	-90°
SRK06-13	7563353.0	432539.3	-0.5	-90°
SRK06-14	7563339.5	432526.1	-0.6	-90°
SRK06-15	7563324.8	432514.9	-0.5	-90°
SRK06-16	7563340.0	432502.3	-0.5	-90°
SRK06-17	7563319.1	432536.8	-0.3	-90°

1. UTM Projection NAD 83 Zone 13.
2. Negative values represent collar elevation below survey grid datum.

Drilling results are summarized in a series of drill logs, included as Appendix A. Drill core photos are included as Appendix B. A generalized profile through the drill holes (Figure 3) displays the interpreted stratigraphy along the proposed jetty centreline. This profile includes drill holes from the April 2004 drill program (SRK 2004), and the bathymetric contours are based on a survey done by Frontier Geosciences in 2003 (Frontier Geosciences 2003).

3.2 Foundation Conditions

3.2.1 SRK06-11

The sea ice at this hole was about 1.07 m thick. The sea ice was frozen to the bed sediments. The surface sediments consist of a layer of sand and gravel. The drill system used for this program in not well suited to penetrate this type of stratigraphy, and as a result the hole was terminated, before the base of the sand zone was found. Sample recovery in this hole was 100%.

3.2.2 SRK06-12

The sea ice at this hole was about 1.83 m thick, which overlies about 2.13 m of unfrozen seawater. The surface sediments consist of a layer of silty clay up to a depth of 19.20 m. The hole was terminated at this depth after refusal of the drill in sand and gravel. Sample recovery of the silty clay was 100%.

3.2.3 SRK06-13

Sea ice at this hole was again about 1.83 m thick, and the unfrozen seawater beneath it was about 2.89 m. From 4.72 m to 19.2 m there was 100% recovery of the same silty clay stratigraphy observed in SRK06-12. The hole was again terminated at this depth due to refusal on sand and gravel.

3.2.4 SRK06-14

This hole encountered sea ice of about 1.83 m, overlying about 2.74 m of unfrozen seawater. Between 4.57 m and 17.68 m the same layer of silty clay was encountered as in previous holes, before reaching a 0.39 m thick layer of sand and gravel. The hole was terminated in bedrock. Sample recovery in this hole was 100%.

3.2.5 SRK06-15

About 1.83 m unfrozen seawater was overlain by about 1.83 m of sea ice in this hole. The upper stratigraphic unit between 3.66 and 6.86 m is silty clay. This overlies a sand layer which was penetrated about 2.28 m before drill refusal. Sample recovery in this hole was 100%.

3.2.6 SRK06-16

The sea ice at this hole was about 1.83 m thick, overlying about 2.74 m of unfrozen seawater. The silty clay in this hole was found between 4.57 m to 17.68 m, overlying a sand layer. Drill refusal was encountered about 0.46 m into the sand layer. Sample recovery in this hole was again 100%.

3.2.7 SRK06-17

Sea ice in this hole was only about 1.37 m thick, and extended directly onto the sediments. The surface sediments consist of a layer of sand and gravel from 1.37 m to 3.81 m underlain by 9.91 m of silty clay. Drill refusal was encountered at the base of the silty clay layer; however, it was not clear whether refusal was due to the presence of bedrock or frozen soil. Ice lenses were observed in the silty clay between 9.60 m and 13.72 m, with the ice content about 10%. Recovery in this hole was 100%.

3.3 Laboratory Testing Results

13 samples were subjected to basic foundation indicator testing, with the primary results summarized in Table 4. Four of these samples were subjected to more specialized geotechnical testing, the results of which are summarized in Table 5. Complete laboratory data sheets are included as Appendix C.

Table 4: Results of foundation indicator testing

Sample [ID:Depth (Type)]	Salinity (ppt)	Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
SRK06-11-01: 1.1m (SP)	89	19.2	N/A	N/A	N/A
SRK06-12-02: 7.9m (CL)	-	45.5	27	16	11
SRK06-12-03: 18.6m (CL)	-	46.5	40	20	20
SRK06-13-01: 5.5m (CL)	-	43.8	38	21	17
SRK06-13-02: 18.6m (CL)	-	49.2	42	22	20
SRK06-14-02: 6.9m (CL)	-	43.3	36	20	16
SRK06-15-01: 3.9m (ML)	-	27.2	19	16	3
SRK06-15-02: 7.0m (CL)	-	43.3	36	20	16
SRK06-16-01: 5.5m (CL)	-	37.3	29	17	12
SRK06-16-02: 8.3m (CL)	-	70.7	48	22	26
SRK06-17-01: 2.8m (SP)	-	20.3	N/A	N/A	N/A
SRK06-17-02: 8.0m (CL)	-	35.7	34	19	15
SRK06-17-03: 12.8m (CL)	-	43.4	41	22	19

Table 5: Results of specialized geotechnical testing

Sample [ID:Depth (Type)]	Specific Gravity	Wet Density (Mg/m^3)	Dry Density (Mg/m^3)	Void Ratio	Peak Stress (kPa)
SRK06-12-02: 7.9m (CL)	-	1.918	1.329	-	9.6
SRK06-14-02: 6.9m (CL)	2.703	1.887	1.390	0.945	-
SRK06-16-02: 8.3m (CL)	-	2.076	1.329	-	16.8
SRK06-17-03: 12.8m (CL)	-	2.128	1.576	0.525	-

4 Discussion

Figure 3 shows a longitudinal profile along the proposed jetty centerline, which shows the stratigraphy inferred from the recent drilling results together with those from the April 2004 geotechnical drilling program (SRK 2004).

Based on the current drill hole results is would appear as if the deep sand pocket observed in SRK45 during the April 2004 program, which was always questioned, may not be present. Based on the profile found in SRK06-17, it would appear as if there is a thin layer of sand that occasionally overlie the silt and clay layer; however, it is not likely to be more than a couple of meters thick and is likely present as a result of continuously moving shoreline processes.

The laboratory testing carried out on the silt and clay zone confirm results previously obtained during the April 2005 vane shear testing program. This investigation confirms that the design parameters used for the proposed jetty design (SRK 2005a) is appropriate.

This report, **“Phase III Foundation Investigation Proposed Roberts Bay Jetty Location, Doris North Project, Nunavut, Canada”**, has been prepared by SRK Consulting (Canada) Inc.

Prepared by:

Alvin Tong, E.I.T.
Staff Engineer

Reviewed by:

Maritz Rykaart, Ph.D., P.Eng.
Principal Engineer

5 References

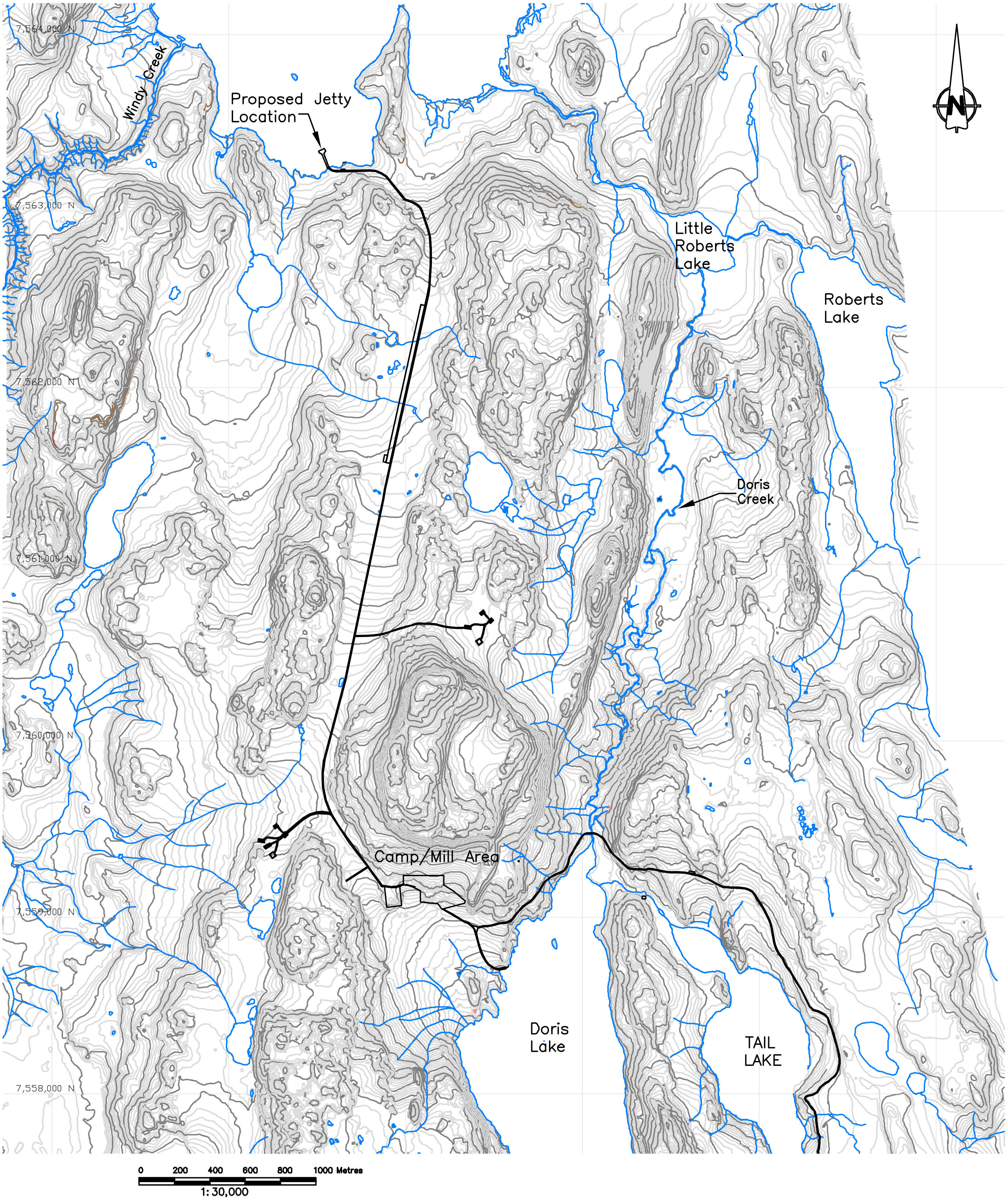
Frontier Geosciences Inc., 2003. *Report on Marine Bathymetry Survey, Proposed Roberts Bay Docking Facilities, Cambridge Bay Area, Nunavut.* Report submitted to SRK Consulting, September 2003.

SRK Consulting (Canada) Inc., 2004. *Phase I Foundation Investigation, Proposed Roberts Bay Jetty Location, Doris North Project, Nunavut, Canada.* Report submitted to Miramar Hope Bay Ltd., Project No. 1CM014.02, April.

SRK Consulting (Canada) Inc., 2005a. *Preliminary Jetty Design, Doris North Project, Hope Bay, Nunavut, Canada.* Report submitted to Miramar Hope Bay Ltd., Project No. 1CM014.006, October.

SRK Consulting (Canada) Inc., 2005b. *Phase II Foundation Investigation, Proposed Roberts Bay Jetty Location, Doris North Project, Nunavut, Canada.* Report submitted to Miramar Hope Bay Ltd., Project No. 1CM014.04-0110, May.

Figures



SRK Consulting
Engineers and Scientists
Vancouver

SRK JOB NO.: 1CM014.008
FILE NAME: SITE-MAP.dwg

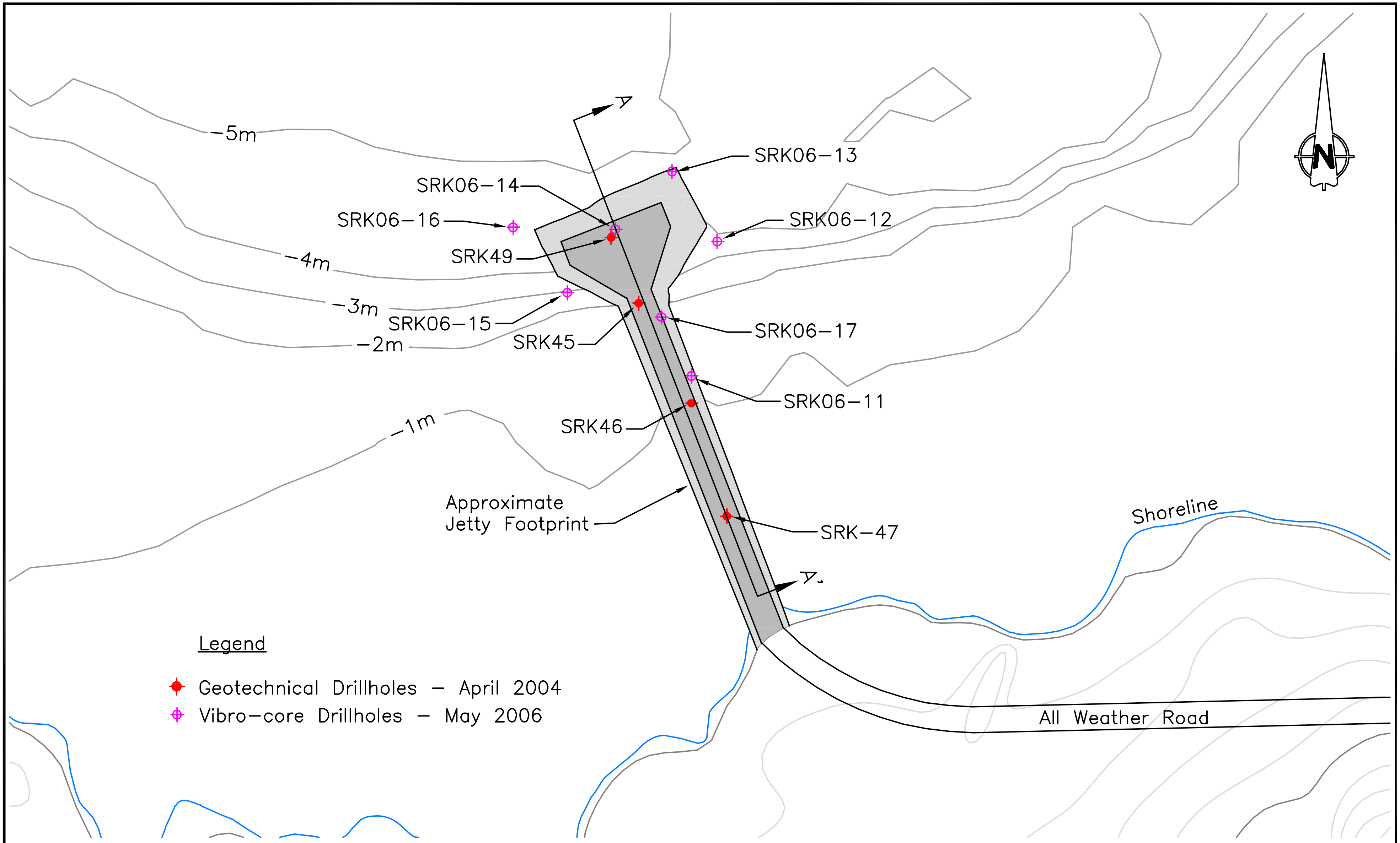
MIRAMAR HOPE BAY LIMITED

Doris North Project

Doris North Project
Phase III Jetty Design

Site Layout and Jetty Location

DATE:	APPROVED:	FIGURE:
Aug. 2006	EMR	1



0 10 20 30 40 50 Metres
1:1250

Contour Interval = 1m
UTM Projection: NAD83 Zone 13
Bathymetry: Frontier Geosciences Inc.
(2003)

SRK Consulting
Engineers and Scientists
Vancouver

SRK JOB NO.: 1CM014.008
FILE NAME: jetty plan.dwg

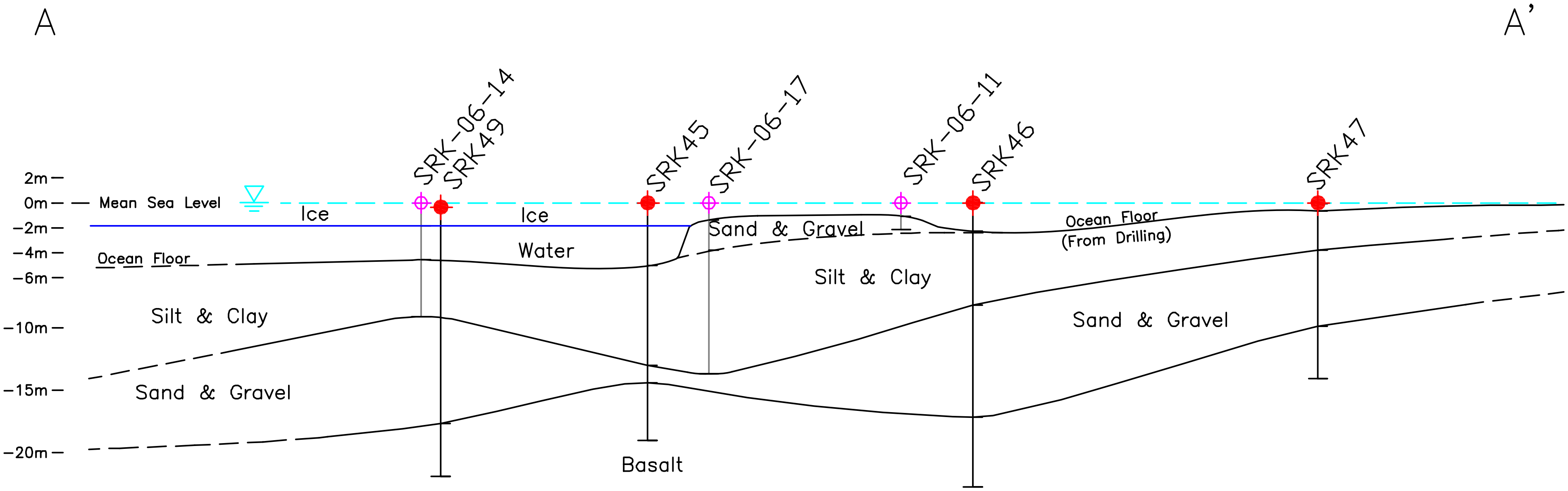
MIRAMAR HOPE BAY LIMITED

Doris North Gold Project

Doris North Project
Phase III Jetty Design

Drill Hole Locations

DATE:	APPROVED:	FIGURE:
Aug. 2006	EMR	2



Legend

● Geotechnical Drillholes – April 2004

⊕ Vibro-core Drillholes – May 2006

0 5 10 15 20 Metres

1:500

Bathymetry: Frontier Geosciences Inc.
(2003)

SRK Consulting
Engineers and Scientists
Vancouver

SRK JOB NO.: 1CM014.008

FILE NAME: Jetty_Profile.dwg

MIRAMAR HOPE BAY LIMITED

Doris North Gold Project

Doris North Project
Phase III Jetty Design

Jetty Centerline Profile

DATE: Aug. 2006

APPROVED: EMR

FIGURE: 3

Appendix A

Drill Logs

SRK Consulting
Engineers and Scientists

BOREHOLE LOG

PROJECT: Doris North - Detailed Infrastructure Design
LOCATION: Jetty
FILE No: HOPE BAY (1CM014.008)
BORING DATE: 2006-05-18 **TO** 2006-05-18
DIP: 90.00 **AZIMUTH:**
COORDINATES: 7563305.40 N 432543.80 E **DATUM:**

BOREHOLE: SRK06-11
PAGE: 1 **OF** 1
DRILL TYPE:
DRILL:
CASING:

SAMPLE CONDITION

- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

TYPE OF SAMPLER

DC	Diamond core barrel
GS	Grab sample
SS	Split spoon

LABORATORY AND IN SITU TESTS

C	Consolidation
D	Bulk density (kg/m3)
Dr	Specific gravity
Ksat	Saturated hydraulic cond. (cm/s)
Ku	Thermal conductivity Unfrozen (W / m°C)
Kf	Thermal conductivity Frozen (W / m°C)
PS	Particle size analysis

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	STRATIGRAPHY			SAMPLES				LABORATORY and IN SITU TESTS	WATER CONTENT and LIMITS (%)
			ELEVATION - m DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD		W_P W W_L 20 40 60 80 100120
			0.00								
			0.00	ICE							
			-1.07 1.07	SAND and GRAVEL with a trace of silt, well graded, dense	░	SRK06-11-01	☒	100	0		⊙
			-2.13 2.13	END OF BOREHOLE							

DEPTH - ft: 5 10 15 20 25 30 35 40 45 50 55 60 65

DEPTH - m: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19

Z:\06 REFERENCE MATERIALS\geotec.log\templates\loglog SRK m23 HopeBay 20m.sty PLOTTED: 2006-08-30 16:20hrs

SRK Consulting
Engineers and Scientists

BOREHOLE LOG

PROJECT: Doris North - Detailed Infrastructure Design
LOCATION: Jetty
FILE No: HOPE BAY (1CM014.008)
BORING DATE: 2006-05-18 **TO** 2006-05-18
DIP: 90.00 **AZIMUTH:**
COORDINATES: 7563336.73 N 432549.82 E **DATUM:**

BOREHOLE: SRK06-12
PAGE: 1 **OF** 1
DRILL TYPE:
DRILL:
CASING: None

SAMPLE CONDITION
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◨ Core

TYPE OF SAMPLER

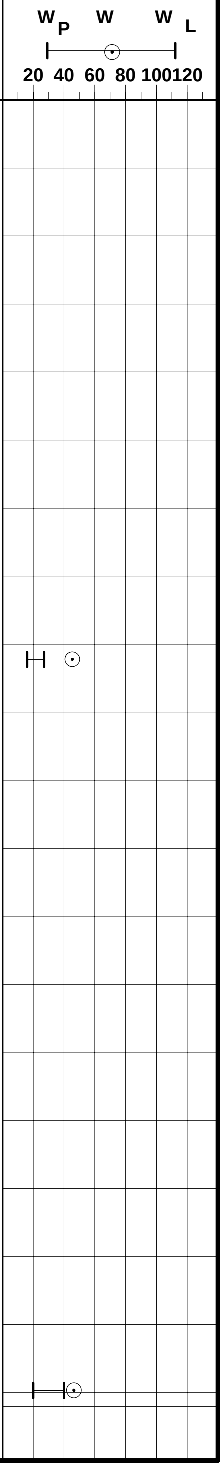
DC	Diamond core barrel
GS	Grab sample
SS	Split spoon

LABORATORY AND IN SITU TESTS

C	Consolidation	Ku	Thermal conductivity Unfrozen (W / m°C)
D	Bulk density (kg/m3)	Kf	Thermal conductivity Frozen (W / m°C)
Dr	Specific gravity	PS	Particle size analysis
Ksat	Saturated hydraulic cond. (cm/s)		

WELL DETAILS & WATER LEVEL - m	ELEVATION - m / DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	LABORATORY and IN SITU TESTS
	0.00 0.00	ICE						
	-1.83 1.83	Water						
	-3.96 3.96	Dark grey silty CLAY, saturated soft, medium plasticity, poorly graded, low consistancy, low dilatency	▨	SRK06-12-01	☒	0	0	
	-5.79 5.79	Same as above but higher consistency and higher silt content	▨	SRK06-12-02	▨	100	0	
	-17.98 17.98	Silty CLAY with a trace of sand, low-med consistency, low dilatency, soft, low, plasticity	▨	SRK06-12-03	☒	100	0	
	-19.20 19.20	EOH END OF BOREHOLE						

WATER CONTENT and LIMITS (%)



DEPTH - ft: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65
DEPTH - m: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19

Z:\06 REFERENCE MATERIALS\geotec.log\templates\loglog SRK m23 HopeBay 20m.sty PLOTTED: 2006-08-30 16:20hrs

SRK Consulting
Engineers and Scientists

BOREHOLE LOG

PROJECT: Doris North - Detailed Infrastructure Design
LOCATION: Jetty
FILE No: HOPE BAY (1CM014.008)
BORING DATE: 2006-05-18 **TO** 2006-05-18
DIP: 90.00 **AZIMUTH:**
COORDINATES: 7563352.97 N 732539.25 E **DATUM:**

BOREHOLE: SRK06-13
PAGE: 1 **OF** 1
DRILL TYPE:
DRILL:
CASING: None

SAMPLE CONDITION
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

TYPE OF SAMPLER

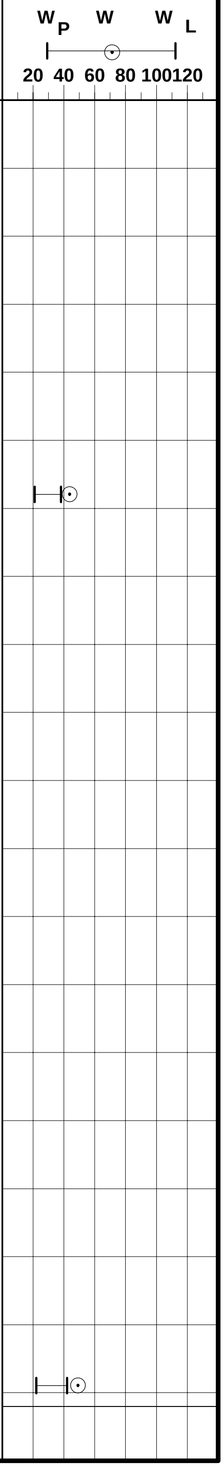
DC	Diamond core barrel
GS	Grab sample
SS	Split spoon

LABORATORY AND IN SITU TESTS

C	Consolidation
D	Bulk density (kg/m3)
Dr	Specific gravity
Ksat	Saturated hydraulic cond. (cm/s)
Ku	Thermal conductivity Unfrozen (W / m°C)
Kf	Thermal conductivity Frozen (W / m°C)
PS	Particle size analysis

ELEVATION - m / DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	LABORATORY and IN SITU TESTS
0.00 0.00	ICE	☒					
-1.83 1.83	Water	≈					
-4.72 4.72	Dark grey silty CLAY, saturated, soft, medium plasticity, poorly graded, low consistancy, low dilatency	▨	SRK06-13-01	▨	100	0	
-6.25 6.25	Same as above but higher consistency and lower dilatency	▨	SRK06-13-02	☒	100	0	
-19.20 19.20	EOH END OF BOREHOLE						

WATER CONTENT and LIMITS (%)



DEPTH - ft: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65
DEPTH - m: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19

WELL DETAILS & WATER LEVEL - m

Z:\06 REFERENCE MATERIALS\geotec.log\templates\loglog SRK m23 HopeBay 20m.sty PLOTTED: 2006-08-30 16:20hrs

SRK Consulting
Engineers and Scientists

BOREHOLE LOG

PROJECT: Doris North - Detailed Infrastructure Design
LOCATION: Jetty Jetty
FILE No: HOPE BAY (1CM014.008)
BORING DATE: 2006-05-17 **TO** 2006-05-17
DIP: 90.00 **AZIMUTH:**
COORDINATES: 7563339.45 N 432526.07 E **DATUM:**

BOREHOLE: SRK06-14
PAGE: 1 **OF** 1
DRILL TYPE:
DRILL:
CASING:

SAMPLE CONDITION

☒	Remoulded
▨	Undisturbed
■	Lost
◧	Core

TYPE OF SAMPLER

DC	Diamond core barrel
GS	Grab sample
SS	Split spoon

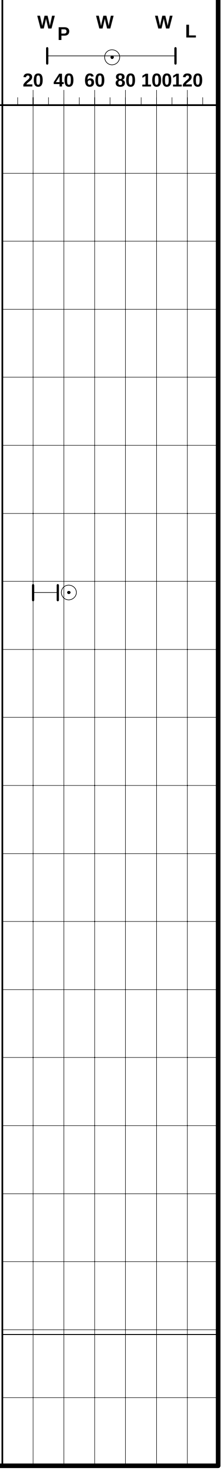
LABORATORY AND IN SITU TESTS

C	Consolidation
D	Bulk density (kg/m3)
Dr	Specific gravity
Ksat	Saturated hydraulic cond. (cm/s)
Ku	Thermal conductivity Unfrozen (W / m°C)
Kf	Thermal conductivity Frozen (W / m°C)
PS	Particle size analysis

WELL DETAILS & WATER LEVEL - m	ELEVATION - m / DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	LABORATORY and IN SITU TESTS
	0.00							
	0.00	ICE						
	-1.83 1.83	Water						
	-4.57 4.57	Dark grey silty CLAY, saturated, soft, medium plasticity, poorly graded, low consistency, low dilatency. Reduced in core length expected due to consolidation from drilling vibrations	▨	SRK06-14-01	☒	0	0	
			▨	SRK04-14-02	▨	100	0	
	-9.14 9.14	Same as above but ligher in colour	▨					
	-17.68 17.68	SAND and GRAVEL, loose, well graded, saturated		SRK06-14-03	☒	0	0	
	-18.07 18.07	EOH EOH END OF BOREHOLE						

DEPTH - ft: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65
DEPTH - m: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19

WATER CONTENT and LIMITS (%)



SRK Consulting
Engineers and Scientists

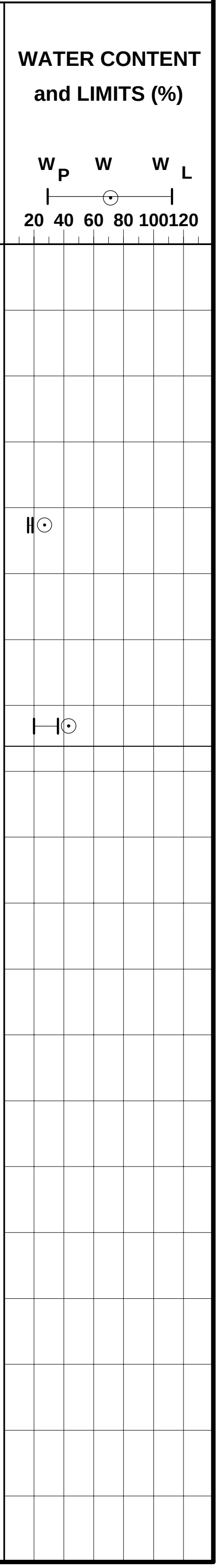
BOREHOLE LOG

PROJECT: Doris North - Detailed Infrastructure Design		**BOREHOLE:** SRK06-15	
LOCATION: Jetty		**PAGE:** 1	**OF** 1
FILE No: HOPE BAY (1CM014.008)		**DRILL TYPE:**	
BORING DATE: 2006-05-19	**TO** 2006-05-19	**DRILL:**	
DIP: 90.00	**AZIMUTH:**	**CASING:**	
COORDINATES: 7563324.81 N 432514.88 E	**DATUM:**		

SAMPLE CONDITION	**TYPE OF SAMPLER**	**LABORATORY AND IN SITU TESTS**	
☒ Remoulded	DC Diamond core barrel	C Consolidation	Ku Thermal conductivity Unfrozen (W / m°C)
▨ Undisturbed	GS Grab sample	D Bulk density (kg/m3)	Kf Thermal conductivity Frozen (W / m°C)
■ Lost	SS Split spoon	Dr Specific gravity	PS Particle size analysis
◧ Core		Ksat Saturated hydraulic cond. (cm/s)	

WELL DETAILS & WATER LEVEL - m	STRATIGRAPHY			SAMPLES				LABORATORY and IN SITU TESTS
	ELEVATION - m DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	
	0.00 0.00	ICE						
	-1.83 1.83	Water						
	-3.66 3.66	Dark grey silty CLAY, saturated, soft, medium plasticity, poorly graded, low consistency, low dilatency	▨	SRK06-15-01	▨	100	0	
	-5.18 5.18	Same as above but higher consistency	▨	SRK06-15-03	☒	0	0	
	-6.86 6.86	SAND with some gravel, loose to dense, well graded, gravel deposited in layers, saturated	⁘	SRK06-15-02	☒	100	0	
	-7.62 7.62	EOH END OF BOREHOLE						

DEPTH - ft: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65
DEPTH - m: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19



Z:\06 REFERENCE MATERIALS\geotec.log\templates\loglog SRK m23 HopeBay 20m.sty PLOTTED: 2006-08-30 16:20hrs

SRK Consulting
Engineers and Scientists

BOREHOLE LOG

PROJECT: Doris North - Detailed Infrastructure Design
LOCATION: Jetty
FILE No: HOPE BAY (1CM014.008)
BORING DATE: 2006-05-17 **TO** 2006-05-17
DIP: 90.00 **AZIMUTH:**
COORDINATES: 7563339.95 N 432502.27 E **DATUM:**

BOREHOLE: SRK06-16
PAGE: 1 **OF** 1
DRILL TYPE:
DRILL:
CASING:

SAMPLE CONDITION
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

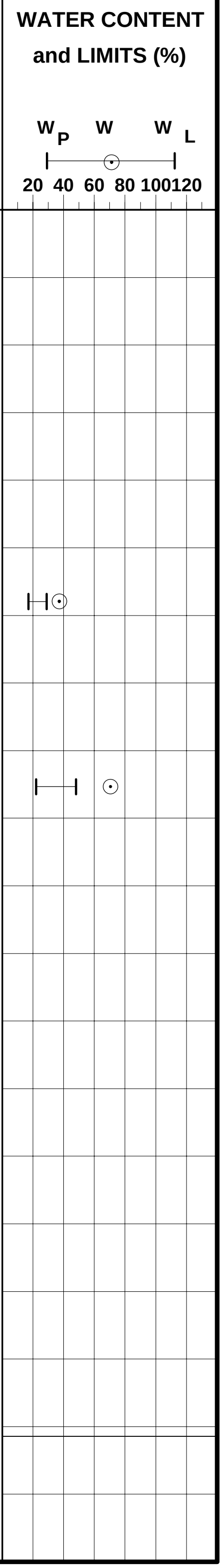
TYPE OF SAMPLER

DC	Diamond core barrel
GS	Grab sample
SS	Split spoon

LABORATORY AND IN SITU TESTS

C	Consolidation	Ku	Thermal conductivity Unfrozen (W / m°C)
D	Bulk density (kg/m3)	Kf	Thermal conductivity Frozen (W / m°C)
Dr	Specific gravity	PS	Particle size analysis
Ksat	Saturated hydraulic cond. (cm/s)		

DEPTH - m	WELL DETAILS & WATER LEVEL - m	ELEVATION - m / DEPTH - m	DESCRIPTION	SYMBOL	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	LABORATORY and IN SITU TESTS
		0.00 0.00	ICE						
		-1.83 1.83	Water						
		-4.57 4.57	Dark grey silty CLAY, saturated, soft, medium plasticity, poorly graded, low consistency, low dilatency	▨					
					SRK06-16-01	☒	100	0	
					SRK06-16-02	▨	100	0	
		-17.68 17.68	SAND, loose, poorly graded, saturated		SRK06-16-03	☒	0	0	
		-17.98 17.98	SAND and GRAVEL, loose, well graded, saturated						
		-18.14 18.14	EOH END OF BOREHOLE						



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SRK Consulting
Engineers and Scientists

BOREHOLE LOG

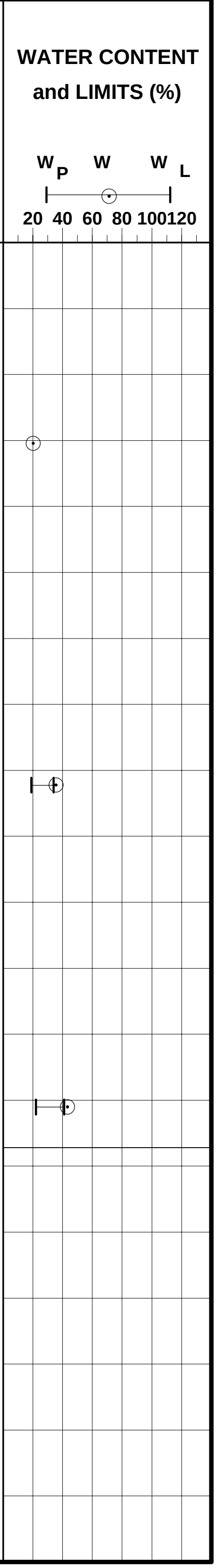
PROJECT: Doris North - Detailed Infrastructure Design
LOCATION: Jetty
FILE No: HOPE BAY (1CM014.008)
BORING DATE: 2006-05-18 **TO** 2006-05-18
DIP: 90.00 **AZIMUTH:**
COORDINATES: 7563319.13 N 432536.83 E **DATUM:**

BOREHOLE: SRK06-17
PAGE: 1 **OF** 1
DRILL TYPE:
DRILL:
CASING:

SAMPLE CONDITION	**TYPE OF SAMPLER**	**LABORATORY AND IN SITU TESTS**	
☒ Remoulded	DC Diamond core barrel	C Consolidation	Ku Thermal conductivity Unfrozen (W / m°C)
▨ Undisturbed	GS Grab sample	D Bulk density (kg/m3)	Kf Thermal conductivity Frozen (W / m°C)
■ Lost	SS Split spoon	Dr Specific gravity	PS Particle size analysis
◧ Core		Ksat Saturated hydraulic cond. (cm/s)	

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	STRATIGRAPHY ELEVATION - m / DEPTH - m	DESCRIPTION	SYMBOL	SAMPLES TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	LABORATORY and IN SITU TESTS
			0.00 0.00	ICE						
			-1.37 1.37	SAND and GRAVEL with trace of silt, well graded, dense, saturated	⁘	SRK06-17-01	☒	100	0	
			-3.81 3.81	Dark graded silty CLAY, saturated, soft, medium plasticity, poorly graded, low consistency, low dilatency	▨	SRK06-17-02	☒	100	0	
			-9.60 9.60	Same as above but has evidence of ice lenses, Vr, 10% ice	▨	SRK06-17-03	▨	100	0	
			-13.72 13.72	EOH. Driller could not tell if refusal is due to frozen soil or bedrock END OF BOREHOLE						

DEPTH - ft: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65
DEPTH - m: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19



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Appendix B

Drill Core Photos

Photo 1: SRK06-11, 3.5' – 7.0' (1.07m – 2.13m)

Photo 2: SRK06-12, 13.5' – 63.0' (3.96m – 19.20m)

Photo 3: SRK06-13, 15.5’ – 63’ (4.72m – 19.2m)

Photo 4: SRK06-14, 15.0’ – 59.4’ (4.57m – 18.07m)

Photo 5: SRK06-15, 12' – 48' (3.66m – 7.62)

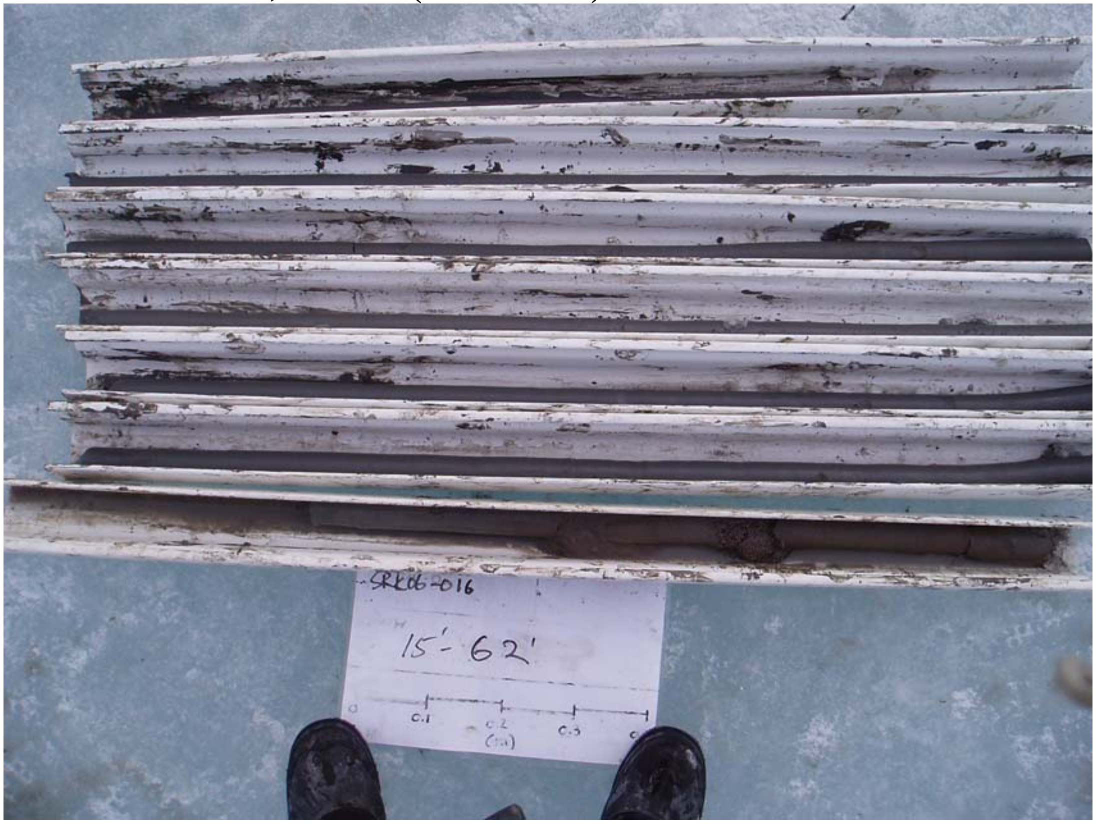


Photo 6: SRK06-16, 15' – 62' (4.57m – 18.14m)

Photo 7: SRK06-17, 4.5’ – 45.0’ (1.37m – 13.72m)

Appendix C

Laboratory Test Results

EBA Engineering Consultants Ltd.

MOISTURE CONTENT TEST RESULTS

Project: SRK 2006 Testing Services
Hope Bay Gold Project
BH No:
Project No.: 1780176
Date Tested: 1-Jun-06
Location: Hope Bay, NT
By: DKKS
Client: SRK Consulting

Test No.	SampleNo.	Depth(m)	Wet+Tare	Dry+Tare	Tare	% Moisture Content
SRK06-01-03	4150-3	N/A	660.9	522.8	12.2	27.0
SRK06-01-06	4150-6	N/A	441.9	285.4	12.5	57.3
SRK06-01-10	4150-10	N/A	707.9	559.7	12.3	27.1
SRK06-02-01	4150-11	N/A	518.4	322.1	12.2	63.3
SRK06-02-06	4150-16	N/A	586.1	405	14.2	46.3
SRK06-02-10	4150-20	N/A	826.9	497.8	13.9	68.0
SRK06-02-13	4150-23	N/A	903.8	750.3	13.9	20.8
SRK06-11-01	4150-25	N/A	705.0	593.4	12.4	19.2
SRK06-12-02	4150-27	N/A	270.2	189.5	12.1	45.5
SRK06-12-03	4150-28	N/A	640.9	441.3	12.4	46.5
SRK06-13-01	4150-29	N/A	292.4	207.0	12.2	43.8
SRK06-13-02	4150-30	N/A	701.6	474.3	12.5	49.2
SRK06-14-02	4150-32	N/A	242.8	173.1	12.2	43.3
SRK06-15-01	4150-34	N/A	288.9	229.8	12.3	27.2
SRK06-15-02	4150-35	N/A	603.9	425.1	12.3	43.3
SRK06-16-01	4150-37	N/A	654.0	479.5	12.2	37.3
SRK06-16-02	4150-38	N/A	176.8	108.6	12.1	70.7
SRK06-17-01	4150-40	N/A	793.9	662.5	13.9	20.3
SRK06-17-02	4150-41	N/A	693.0	514.5	13.9	35.7
SRK06-17-03	4150-42	N/A	262.5	186.8	12.4	43.4

POREWATER SALINITY

Project:	Hope Bay Gold	Sample No.:	SRK06
Project No.:	0701-1780176	Date Tested:	06-06-28
Client:	Miramar Hope Bay Limited	Tested By:	KP

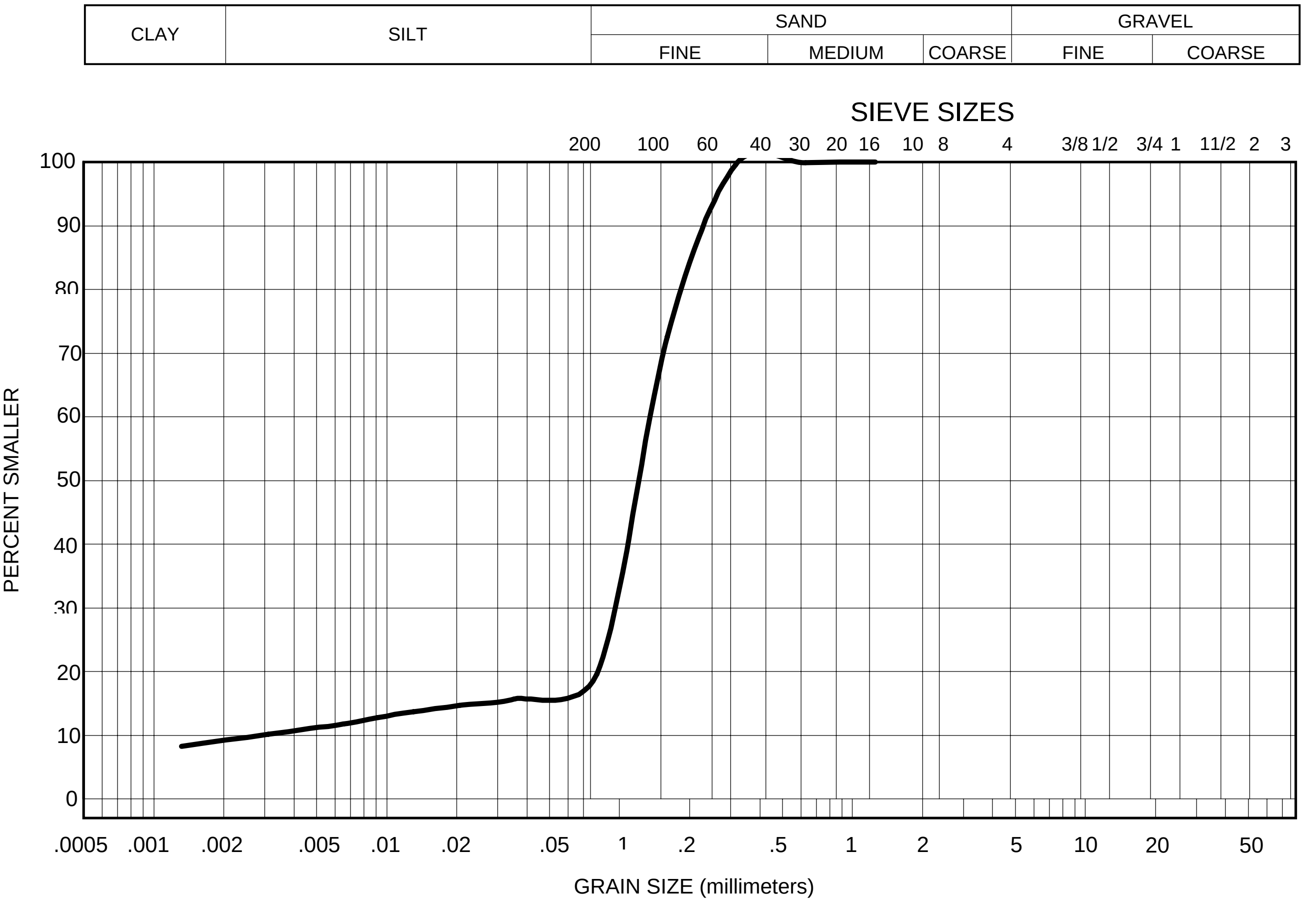
Sample Number	Depth (m)	Salinity (ppt)
01-03		67
01-06		44
01-10		67
02-01		6
02-06		60
02-10		80
02-13		86
11-01		89

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 13-14, 2006
Sample ID: SRK06-11-01
Depth: n/a
Sample Number: n/a
Lab Number: 4150-25
Soil Description: SAND, some silt, trace clay
Natural Moisture Content: 19.2%
Remarks: N.P.

SIEVE, mm	PERCENTAGE PASSING
2.5	
1.25	
0.630	
0.315	100
0.160	73
0.08	20
0.035	16
0.020	15
0.0130	14
0.0092	13
0.0065	12
0.0031	10
0.0013	8



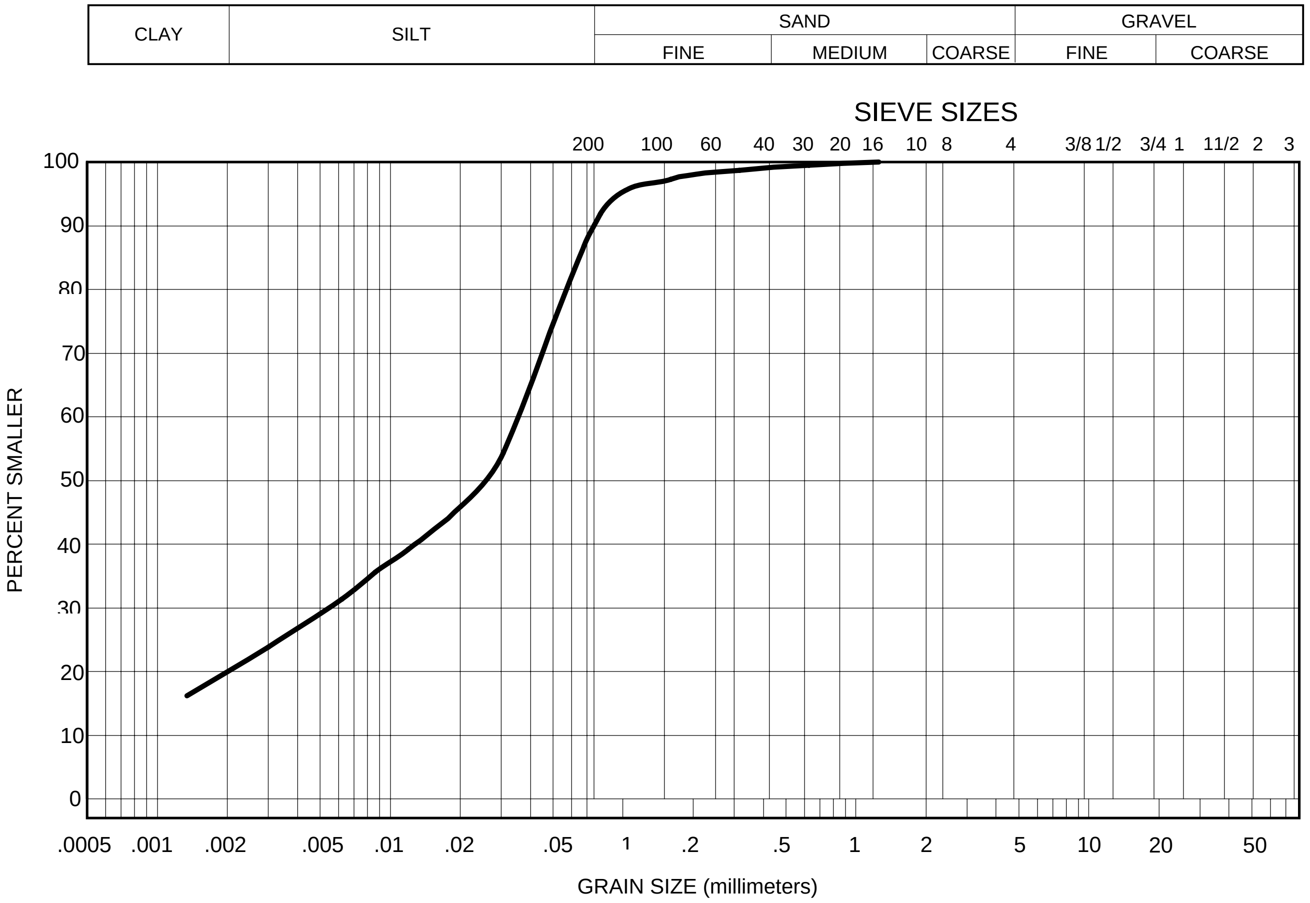
Reviewed By:____________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 12-13, 2006
Sample ID: SRK06-12-02
Depth: n/a
Sample Number: n/a
Lab Number: 4150-27
Soil Description: SILT, some clay, trace sand
Natural Moisture Content: 45.5%
Remarks: LL=27%, PL=16%, PI=11%

SIEVE, mm	PERCENTAGE PASSING
2.5	
1.25	100
0.630	99
0.315	99
0.160	97
0.08	92
0.031	54
0.020	46
0.0120	39
0.0086	36
0.0062	31
0.0030	24
0.0013	16

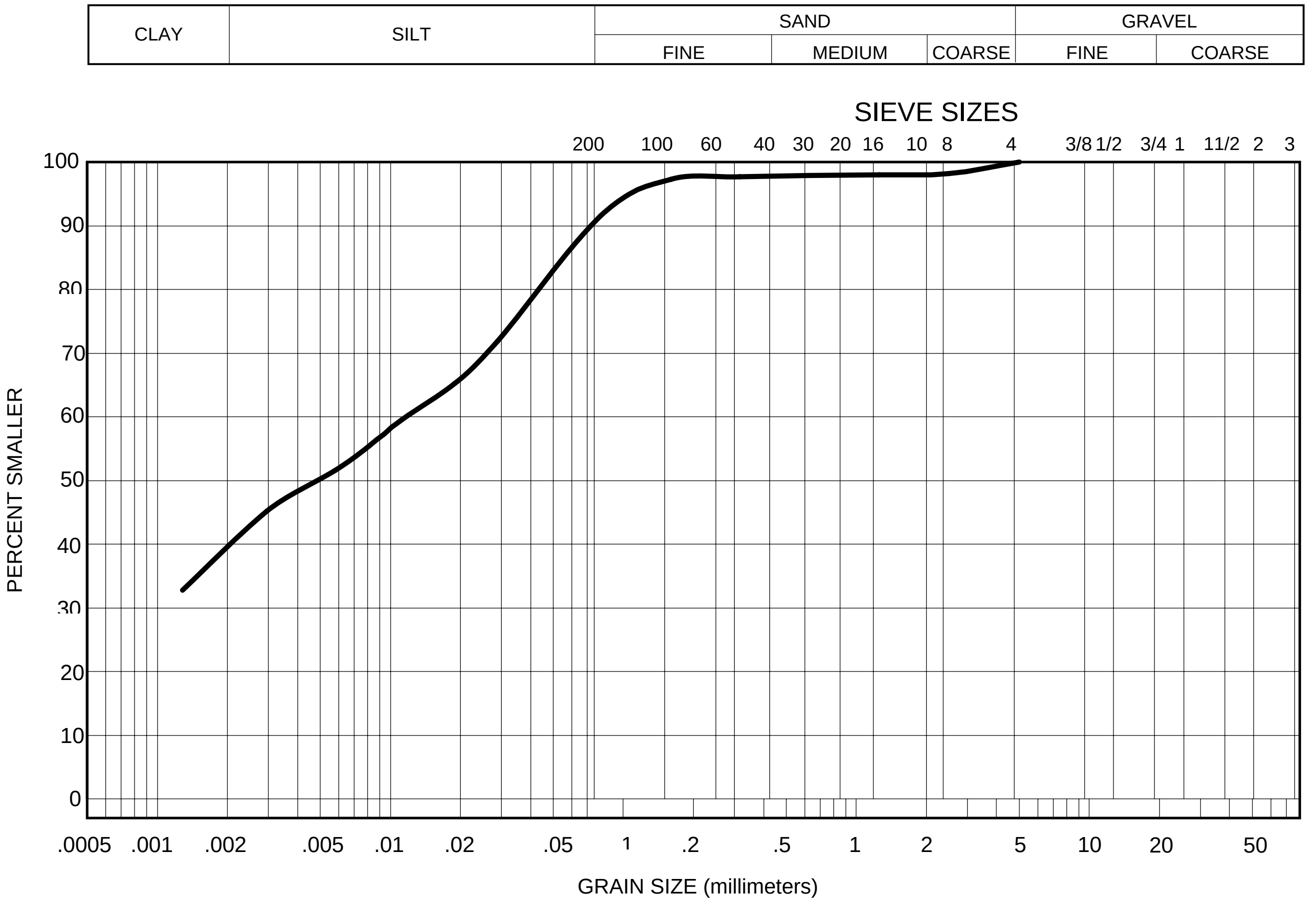


Reviewed By:___________________ P.Eng.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 12-13,14-15, 2006
Sample ID: SRK06-12-03
Depth: n/a
Sample Number: n/a
Lab Number: 4150-28
Soil Description: SILT and CLAY, trace sand
Natural Moisture Content: 46.5%
Remarks: LL=40%, PL=20%, PI=20%

SIEVE, mm	PERCENTAGE PASSING
5.00	100
2.5	98
1.25	98
0.630	98
0.315	98
0.160	97
0.08	92
0.029	72
0.019	65
0.0112	59
0.0081	55
0.0058	52
0.0029	45
0.0013	33



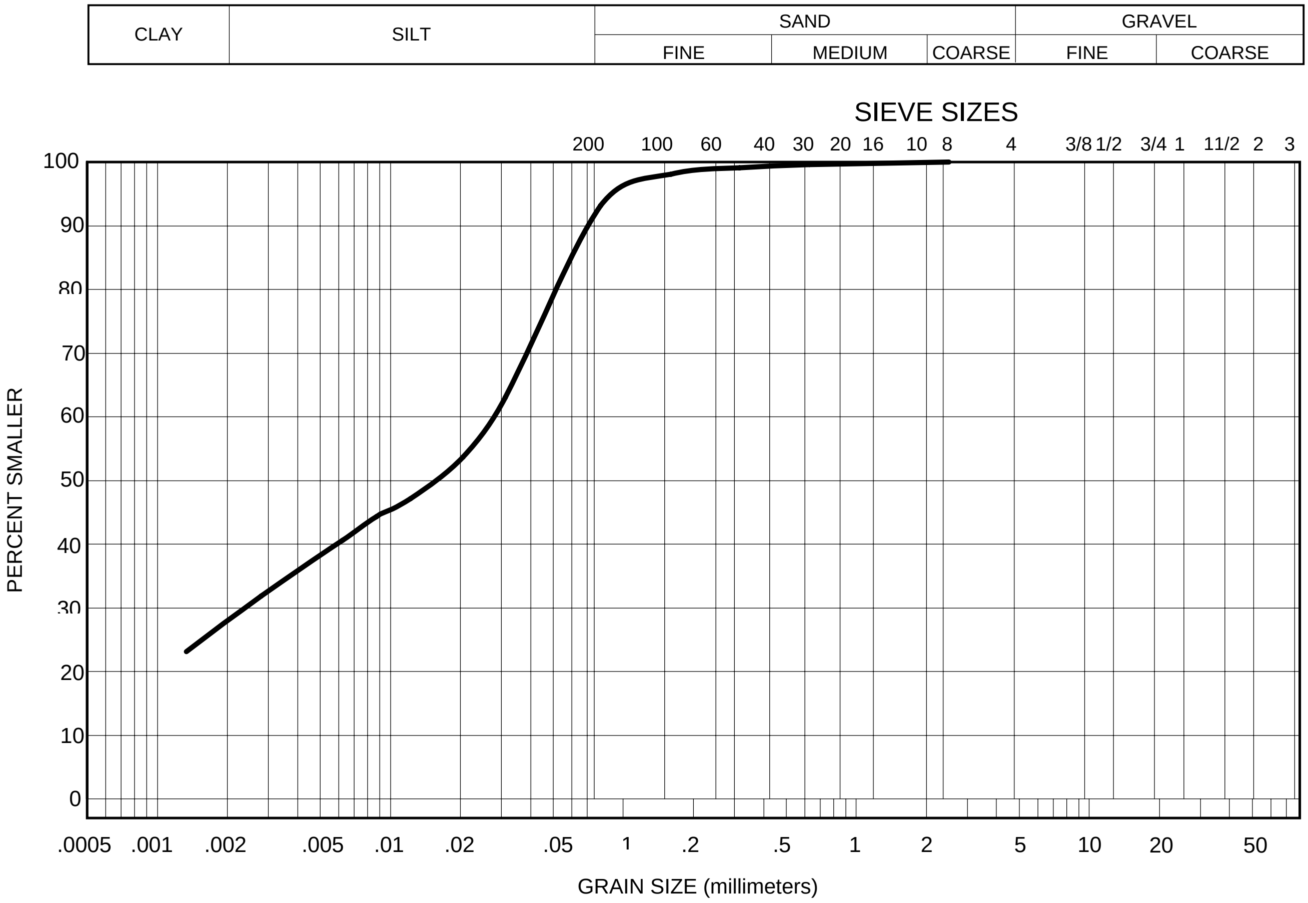
Reviewed By:__________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project.
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 7-9, 2006
Sample ID: SRK06-13-01
Depth: n/a
Sample Number: n/a
Lab Number: 4150-29
Soil Description: SILT, clayey, trace sand
Natural Moisture Content: 43.8%
Remarks: LL=38%, PL=21%, PI=17%

SIEVE	PERCENTAGE PASSING
2.5	
1.25	
0.630	100
0.315	99
0.160	98
0.08	93
0.031	63
0.021	54
0.0122	47
0.0087	44
0.0063	41
0.0028	32
0.0013	23



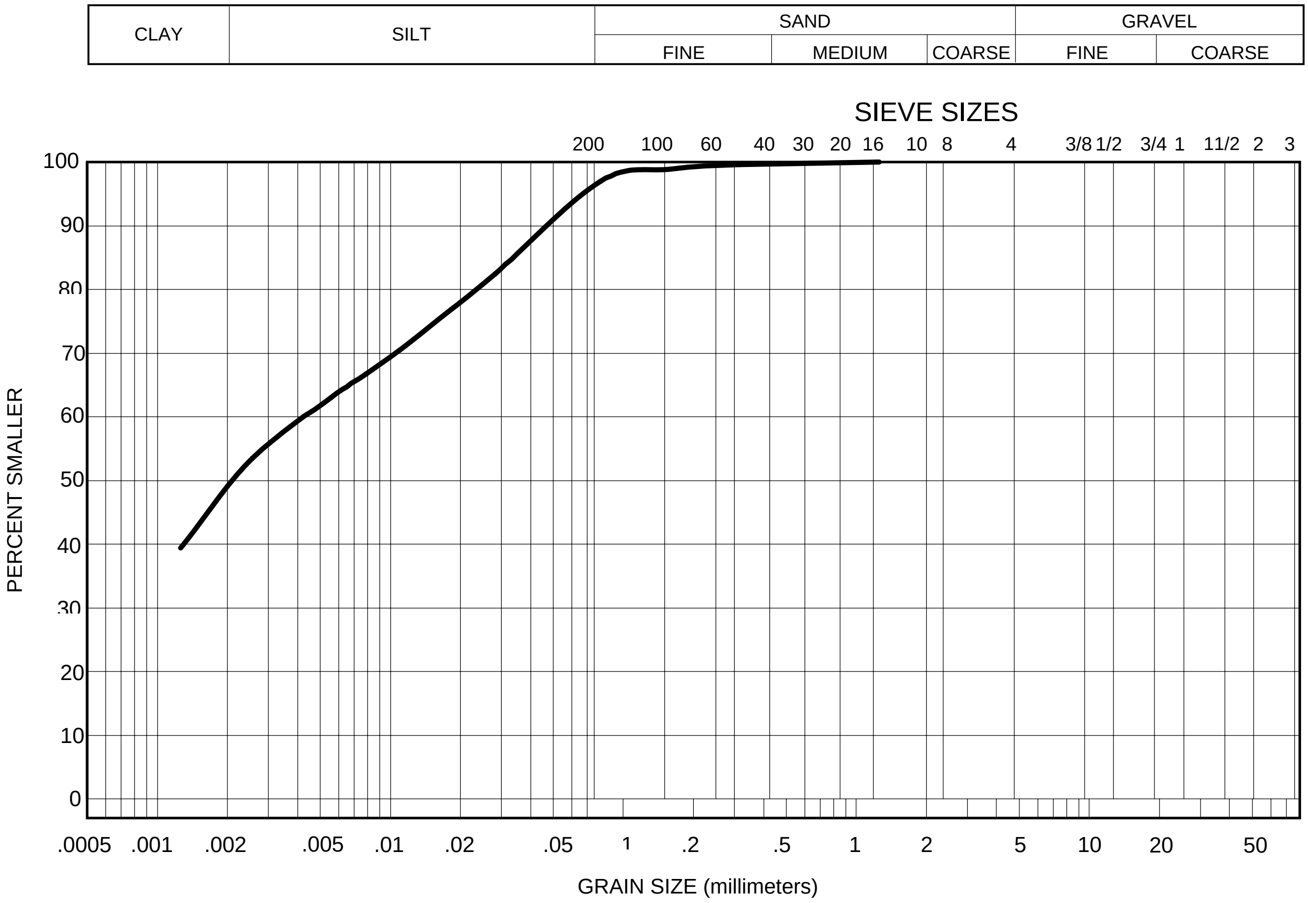
Reviewed By:__________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services. Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 7-9, 2006
Sample ID: SRK06-13-02
Depth: n/a
Sample Number: n/a
Lab Number: 4150-30
Soil Description: CLAY and SILT, trace sand
Natural Moisture Content: 49.2%
Remarks: LL=42%, PL=22%, PI=20%

SIEVE, mm	PERCENTAGE PASSING
2.5	
1.25	
0.630	
0.315	100
0.160	99
0.08	97
0.028	82
0.018	77
0.0109	70
0.0079	67
0.0056	63
0.0026	54
0.0013	39



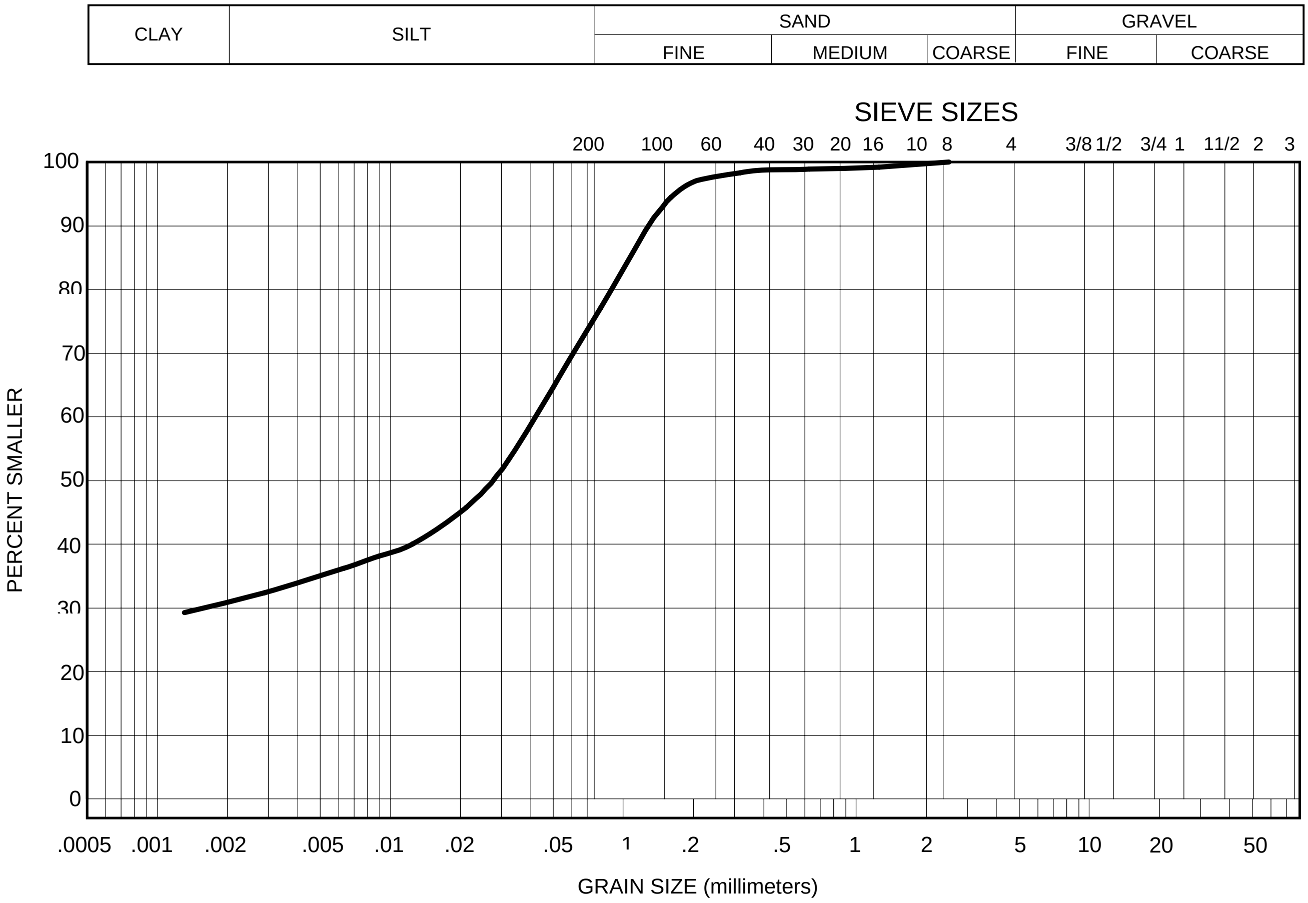
Reviewed By:___________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services. Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr.Alvin Tong
Date Tested: June 7-9,13, 2006
Sample ID: SRK06-14-02
Depth: n/a
Sample Number: n/a
Lab Number: 4150-32
Soil Description: SILT, clayey, sandy
Natural Moisture Content: 43.3%
Remarks: LL=36%, PL=20%, PI=16%

SIEVE, mm	PERCENTAGE PASSING
2.5	100
1.25	99
0.630	99
0.315	98
0.160	94
0.08	77
0.032	53
0.021	46
0.0125	40
0.0089	38
0.0063	36
0.0029	32
0.0013	29



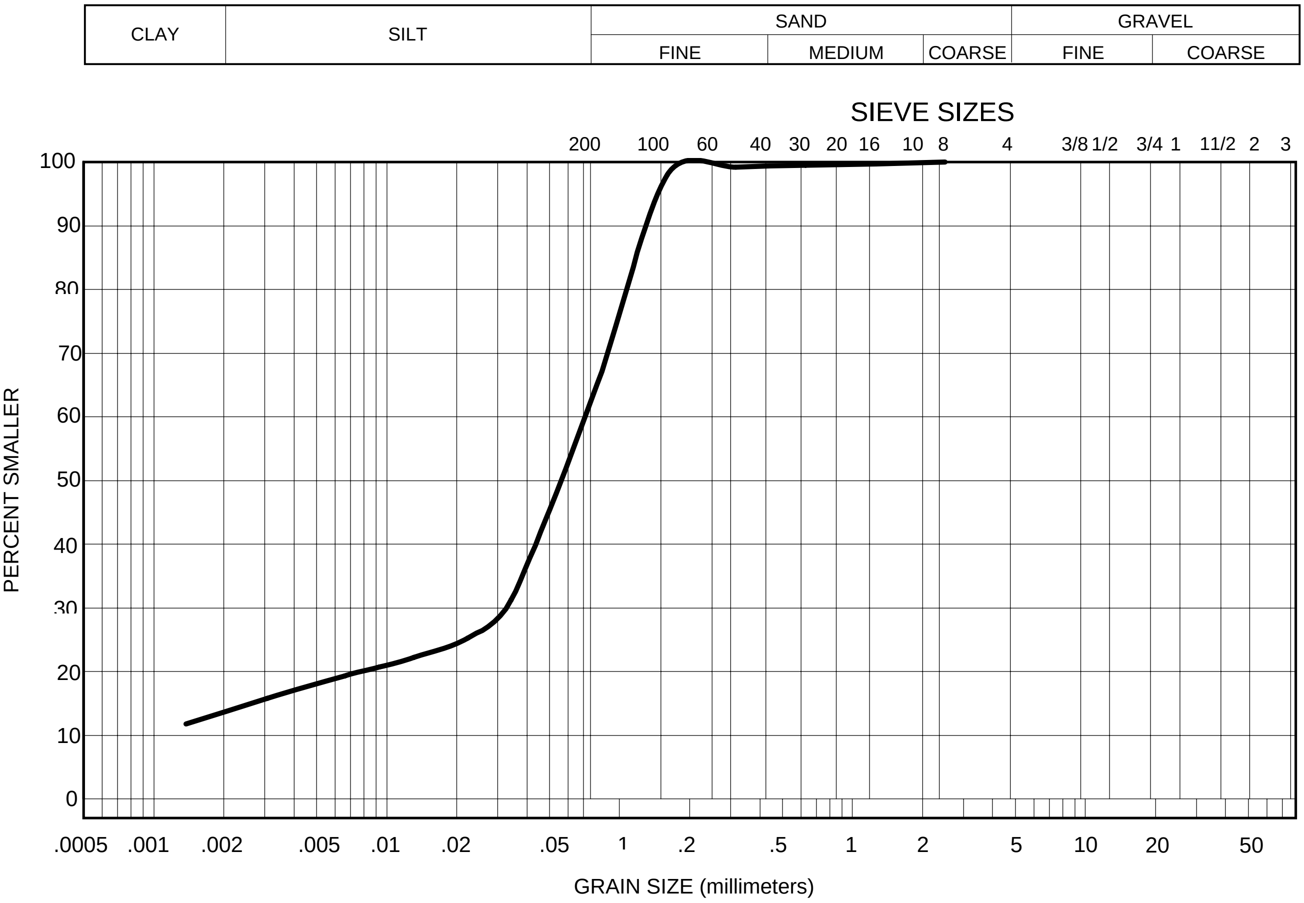
Reviewed By:__________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services. Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr.Alvin Tong
Date Tested: June7-9,13, 2006
Sample ID: SRK06-15-01
Depth: n/a
Sample Number: n/a
Lab Number: 4150-34
Soil Description: SILT, sandy, some clay
Natural Moisture Content: 27.2%
Remarks: LL=19%, PL=16%, PI=3%

SIEVE, mm	PERCENTAGE PASSING
2.5	
1.25	
0.630	100
0.315	99
0.160	98
0.08	65
0.034	31
0.022	25
0.0131	22
0.0093	21
0.0066	19
0.0031	16
0.0014	12



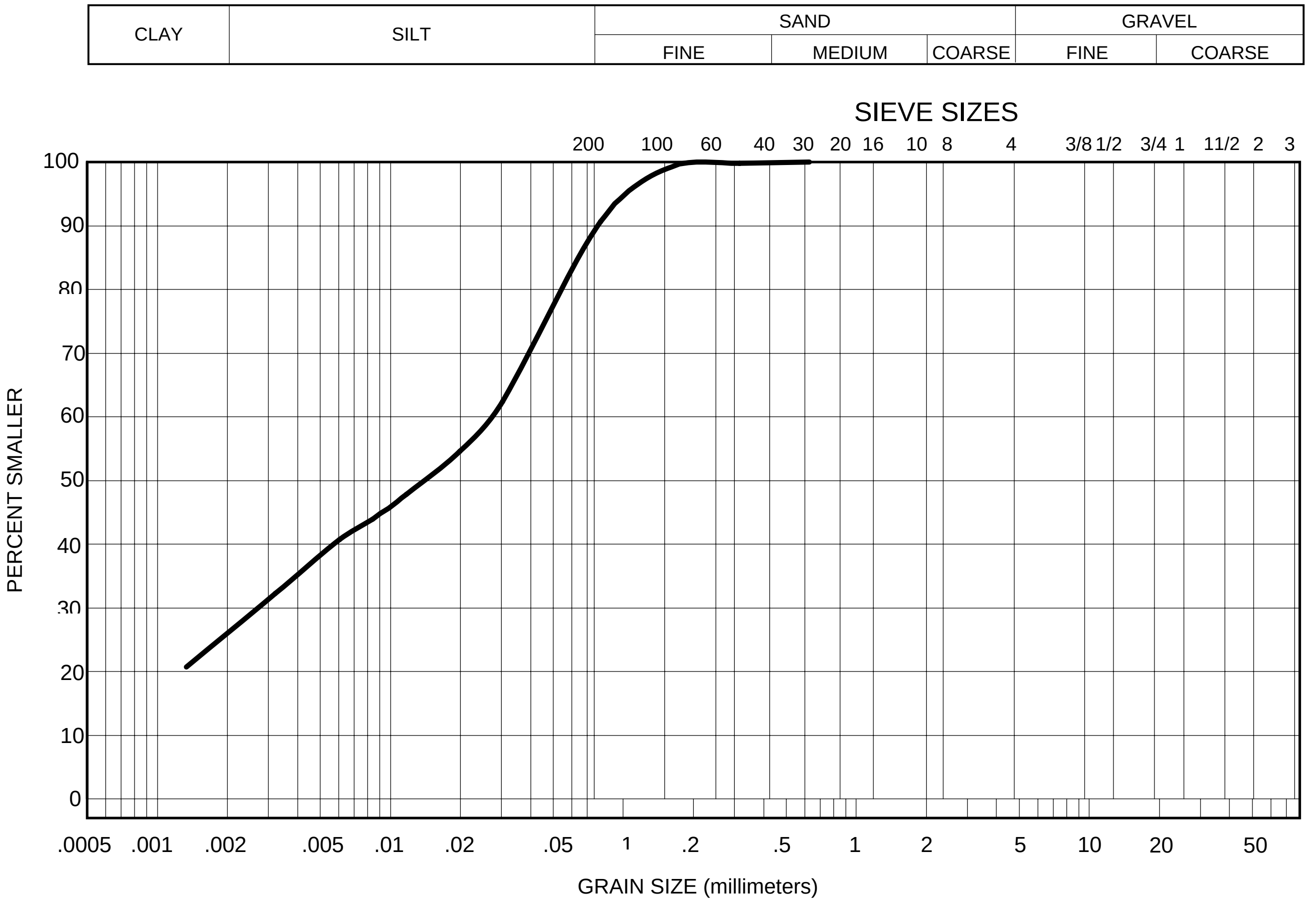
Reviewed By:__________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 10-11,13, 2006
Sample ID: SRK06-15-02
Depth: n/a
Sample Number: n/a
Lab Number: 4150-35
Soil Description: SILT clayey, trace sand
Natural Moisture Content: 43.3%
Remarks: LL=36%, PL=20%, PI=16%

SIEVE	PERCENTAGE PASSING
2.5	
1.25	
0.630	
0.315	100
0.160	99
0.08	91
0.030	62
0.020	55
0.0117	48
0.0084	44
0.0061	41
0.0032	32
0.0013	21

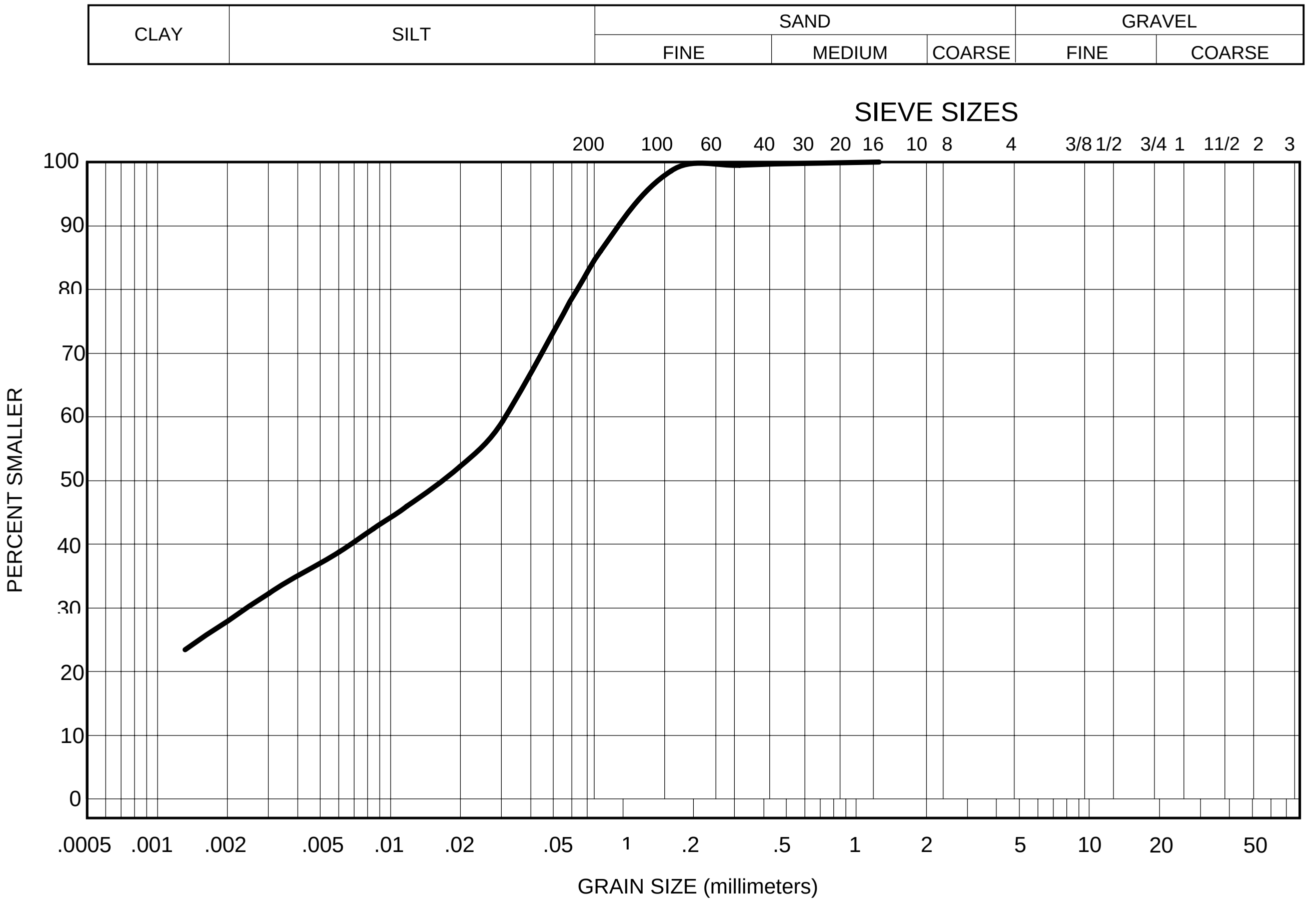


Reviewed By:___________________ P.Eng.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 10-11, 14-15, 2006
Sample ID: SRK06-16-01
Depth: n/a
Sample Number: n/a
Lab Number: 4150-37
Soil Description: SILT, clayey, some sand
Natural Moisture Content: 37.3%
Remarks: LL=29%, PL=17%, PI=12%

SIEVE	PERCENTAGE PASSING
2.5	
1.25	
0.630	100
0.315	99
0.160	99
0.08	86
0.030	59
0.020	52
0.0118	46
0.0084	42
0.0061	39
0.0030	32
0.0013	23



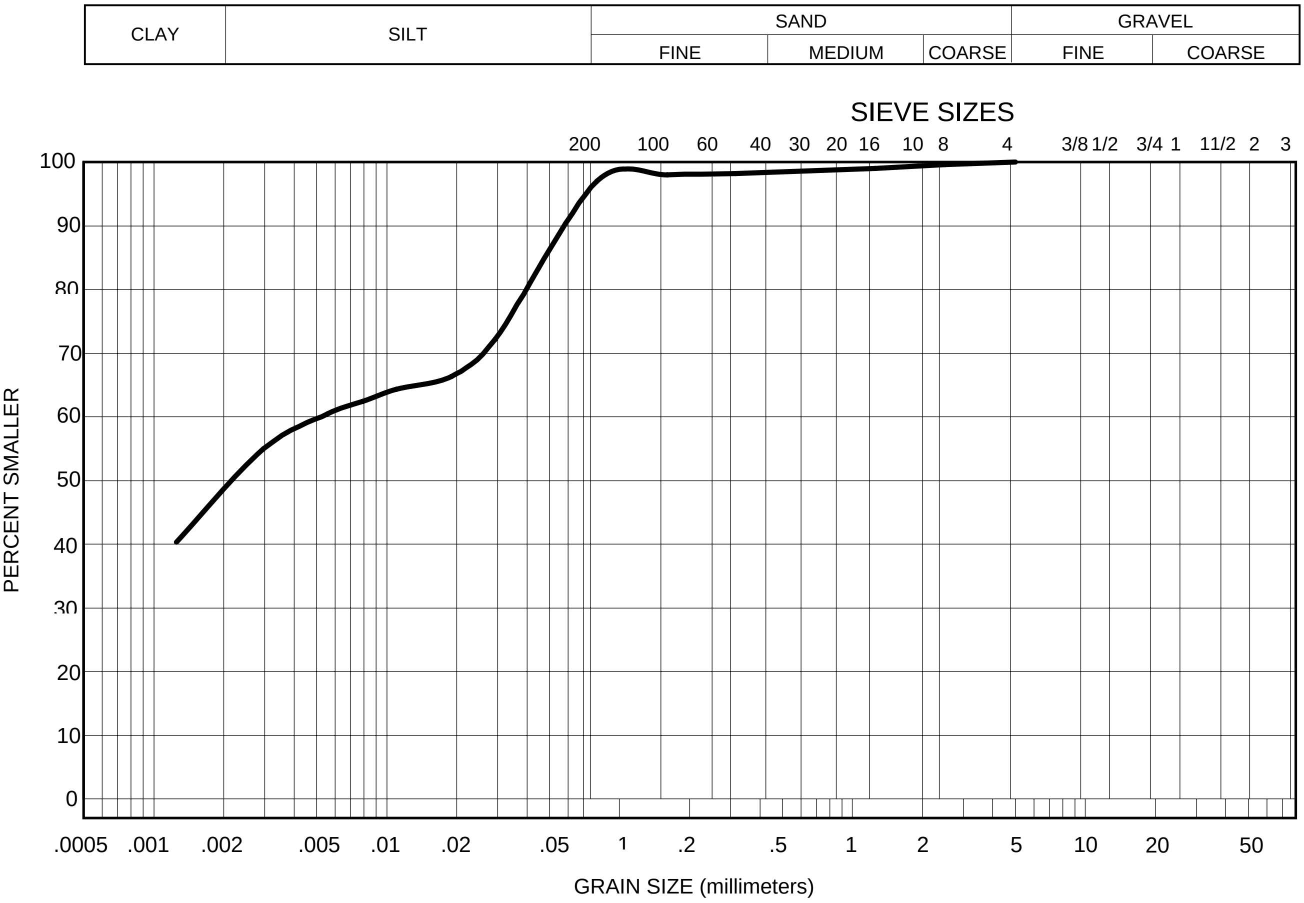
Reviewed By:__________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 10-11,14-15, 2006
Sample ID: SRK06-16-02
Depth: n/a
Sample Number: n/a
Lab Number: 4150-38
Soil Description: SILT and CLAY, trace sand
Natural Moisture Content: 70.7%
Remarks: LL=48%, PL=22%, PI=26%

SIEVE	PERCENTAGE PASSING
2.5	100
1.25	99
0.630	99
0.315	98
0.160	98
0.08	97
0.029	72
0.019	66
0.0111	64
0.0079	62
0.0056	60
0.0030	55
0.0013	40



Reviewed By:___________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project

Project Number: 1780176

Client: SRK Consulting Inc.

Attention: Mr. Alvin Tong

Date Tested: June 10-11, 2006

Sample ID: SRK06-17-01

Depth: n/a

Sample Number: n/a

Lab Number: 4150-40

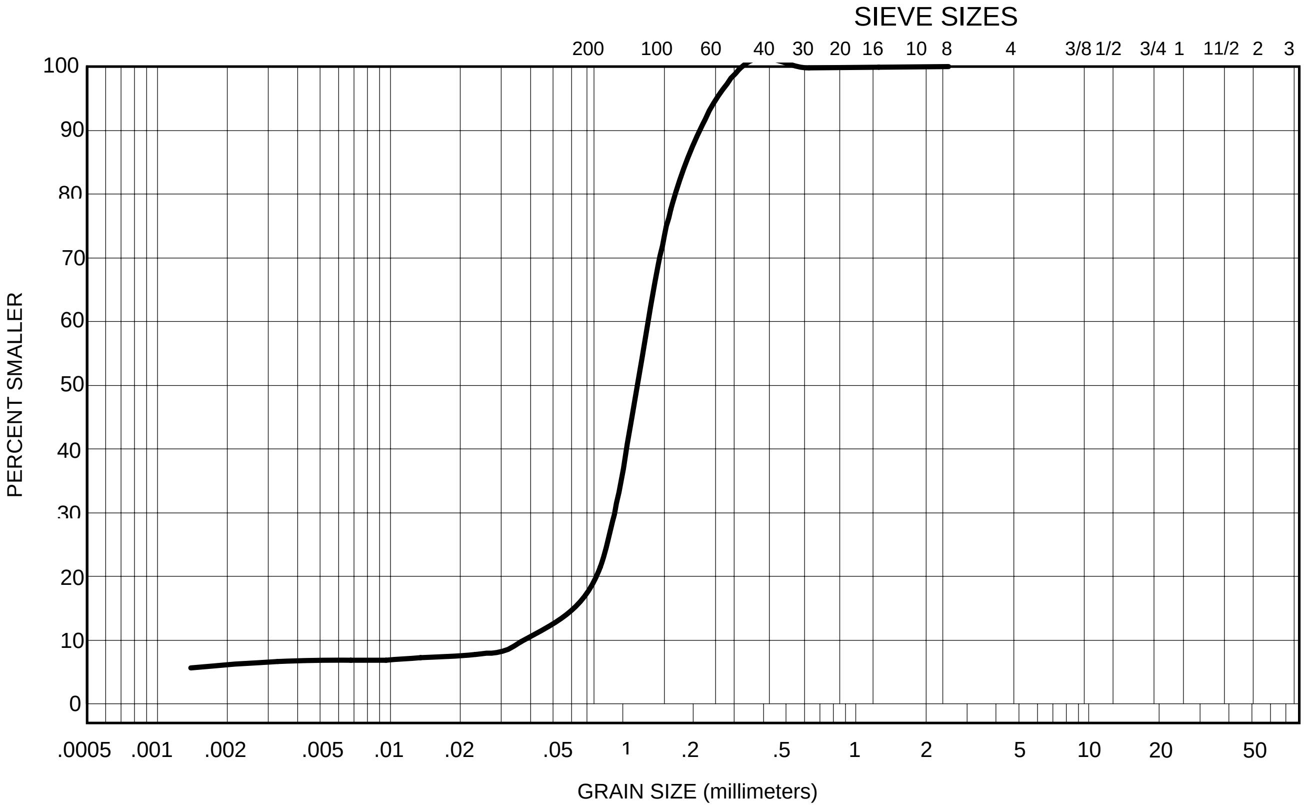
Soil Description: SAND, some silt, trace clay

Natural Moisture Content: 20.3%

Remarks: N.P.

SIEVE	PERCENTAGE PASSING
2.5	
1.25	
0.630	
0.315	100
0.160	78
0.08	21
0.036	10
0.023	8
0.0135	7
0.0096	7
0.0068	7
0.0033	7
0.0014	6

CLAY	SILT	SAND			GRAVEL	
		FINE	MEDIUM	COARSE	FINE	COARSE



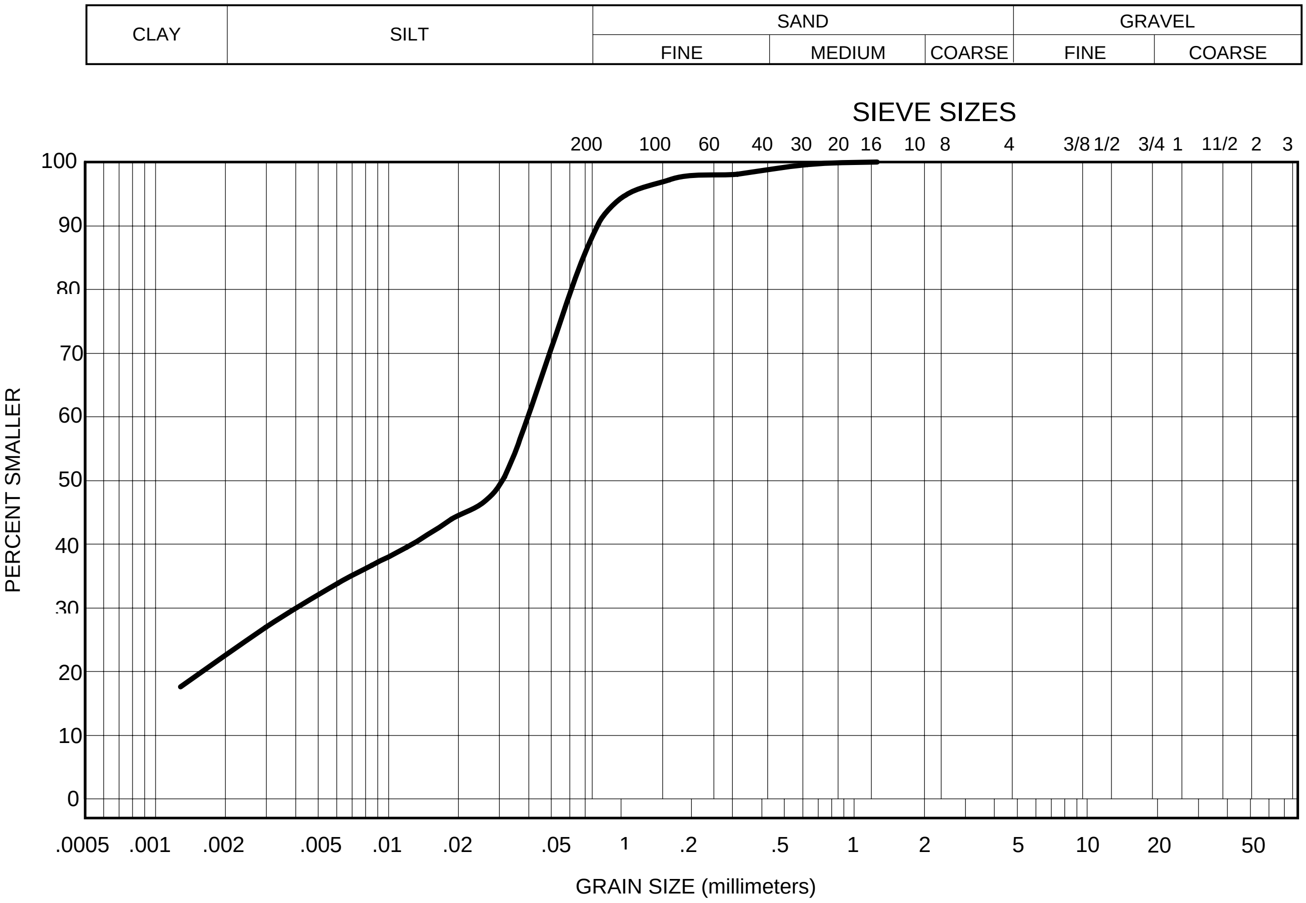
Reviewed By:__________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 13-14,15-16, 2006
Sample ID: SRK06-17-02
Depth: n/a
Sample Number: n/a
Lab Number: 4150-41
Soil Description: SILT, clayey, some sand
Natural Moisture Content: 35.7%
Remarks: LL=34%, PL=19%, PI=15%

SIEVE	PERCENTAGE PASSING
2.5	
1.25	
0.630	100
0.315	98
0.160	97
0.08	90
0.032	51
0.019	44
0.0119	40
0.0085	37
0.0061	34
0.0030	27
0.0013	18



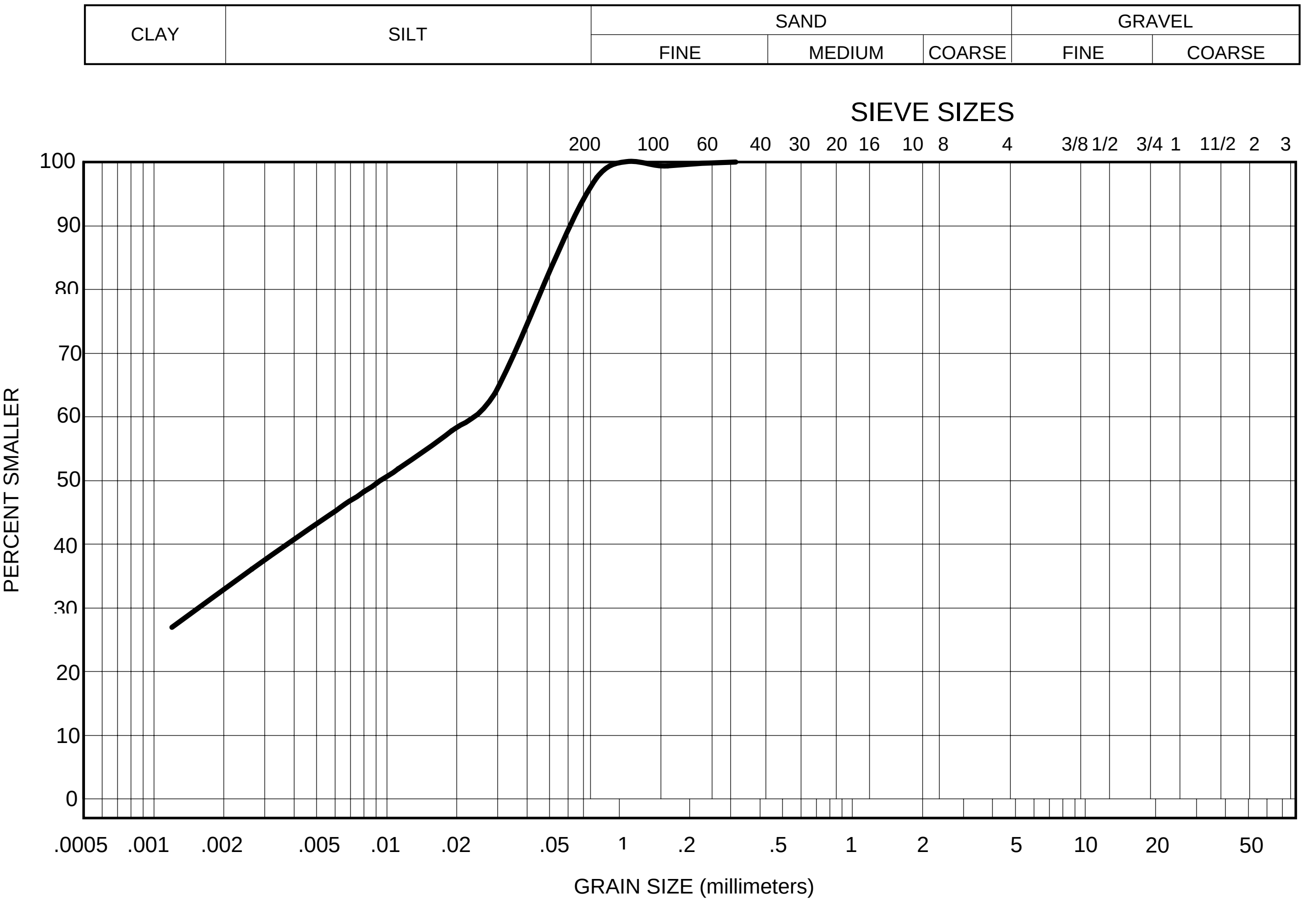
Reviewed By:___________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 14-16, 2006
Sample ID: SRK06-17-03
Depth: n/a
Sample Number: n/a
Lab Number: 4150-42
Soil Description: SILT, clayey, trace sand
Natural Moisture Content: 43.4%
Remarks: LL=41%, PL=22%, PI=19%

SIEVE	PERCENTAGE PASSING
2.5	
1.25	
0.630	
0.315	100
0.160	99
0.08	98
0.029	64
0.019	58
0.0112	52
0.0080	48
0.0057	45
0.0026	36
0.0012	27



Reviewed By:__________________ P.Eng.

EBA Engineering Consultants Ltd.

14535 - 118th AVENUE
EDMONTON, ALBERTA
Phone (403) 451-2121

5664 BURLEIGH CRES. S.E.
CALGARY, ALBERTA
Phone (403) 253-7121

SPECIFIC GRAVITY OF SOIL

(ASTM Designation D854)

Project: Tail Lake and Jetty 2006
Test Hole No.: SRK 06-14-02
Address:
Depth:
Sample No.:
Lab No.:
Project No.: 1780176
Sample Description:
Date Tested: 06.07.20
By: KP

TRIAL	1	2	3
Pycnometer No.	K	C	
Wt. of Soil, Pycnometer & Water (Wb)	174.45	177.59	
Wt. of Pycnometer	58.64	61.82	
Wt. of Dry Soil (Wo)	25.46	25.53	
Temp. of Soil & Water (Tx °C)	20.87	20.72	
Wt. of Pycnometer & Water @ Tx °C (From Calibration Curve)			
Specific Gravity (Gs)	2.701	2.706	
Avg. Specific Gravity	2.703		

$$G_s = \frac{W_o}{W_o + W_a - W_b}$$

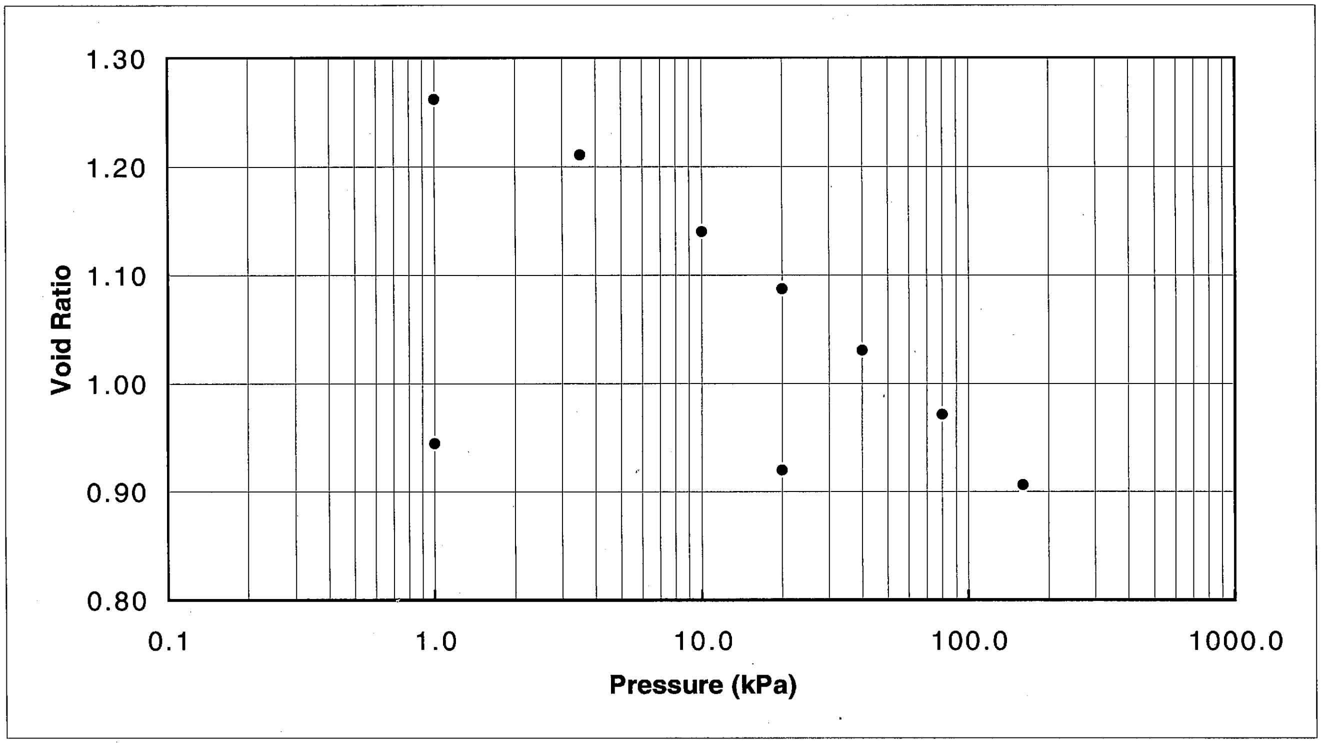
Where:
W_o = Dry wt. of soil
W_a = Wt. of pycnometer & water @ T_x °C (Calibration Curve)
W_b = Wt. of pycnometer, soil & water @ T_x °C
G_s = Specific Gravity of soil sample

Remarks:

CONSOLIDATION TEST **TAIL LAKE AND JETTY 2006**

Project No.:	1780176	Test No.:	C-4
Date Tested:	06-07-20	Sample ID:	SRK06-14-02

	Initial	Final	
Height (mm):	26.47	22.75	
Moisture (%):	48.02	35.74	
Wet Dens. (Mg/m3):	1.769	1.887	
Dry Dens. (Mg/m3):	1.195	1.390	
Void Ratio:	1.2624	0.9447	Spec.Grav.= 2.70
Saturation:	100	100	



P (kPa)	Void Ratio	Cv (m2/yr)	Mv (m2/MN)	K (m/s)
1	1.26			
3.5	1.2114	0.25	9.0140	6.939E-10
10	1.1410	0.41	4.8937	6.180E-10
20	1.0878	0.59	2.4850	4.591E-10
40	1.0306	0.73	1.3697	3.112E-10
80	0.9714	1.15	0.7292	2.602E-10
160	0.9065	1.61	0.4119	2.062E-10
20	0.9196		0.0494	
1	0.9447		0.6869	

SAMPLE INFORMATION

Project: Tail Lake and Jetty 2006

Address:

Project Number: 1780176

Date Tested: 06·07·20 By: S.K.

Test Apparatus: Consolidation

Machine Number: 6

Rate of Strain: mm% / minute

Normal Stress: kPa

Cell Pressure: kPa

Back Pressure: kPa

Head Differential: kPa

Swelling Pressure: kPa

Borehole Number: SRK06 - 14 - 02

Depth:

Test Number: C-4

Sample Description: CLAY, silty, reconstituted.

Sample Description	Diameter (mm)	Height (mm)
1		
2		
3		
4		
Mean	49.96	26.47

$V = 51.89\ cm^3$

	Trimmings	Initial	Final
Tare Number			
Mass of Wet Soil & Tare g		173.05	(84.18) 171.93
Mass of Dry Soil & Tare g			(62.02) 149.82
Mass of Tare g		81.25	6.70
Mass of Dry Soil g			
Mass of Moisture g			
Moisture Content %		48.02	35.74
Wet Density Mg/m^3		1.769	1.887
Dry Density Mg/m^3		1.195	1.390

6.89
6.74
.15

Sketch and Remarks:

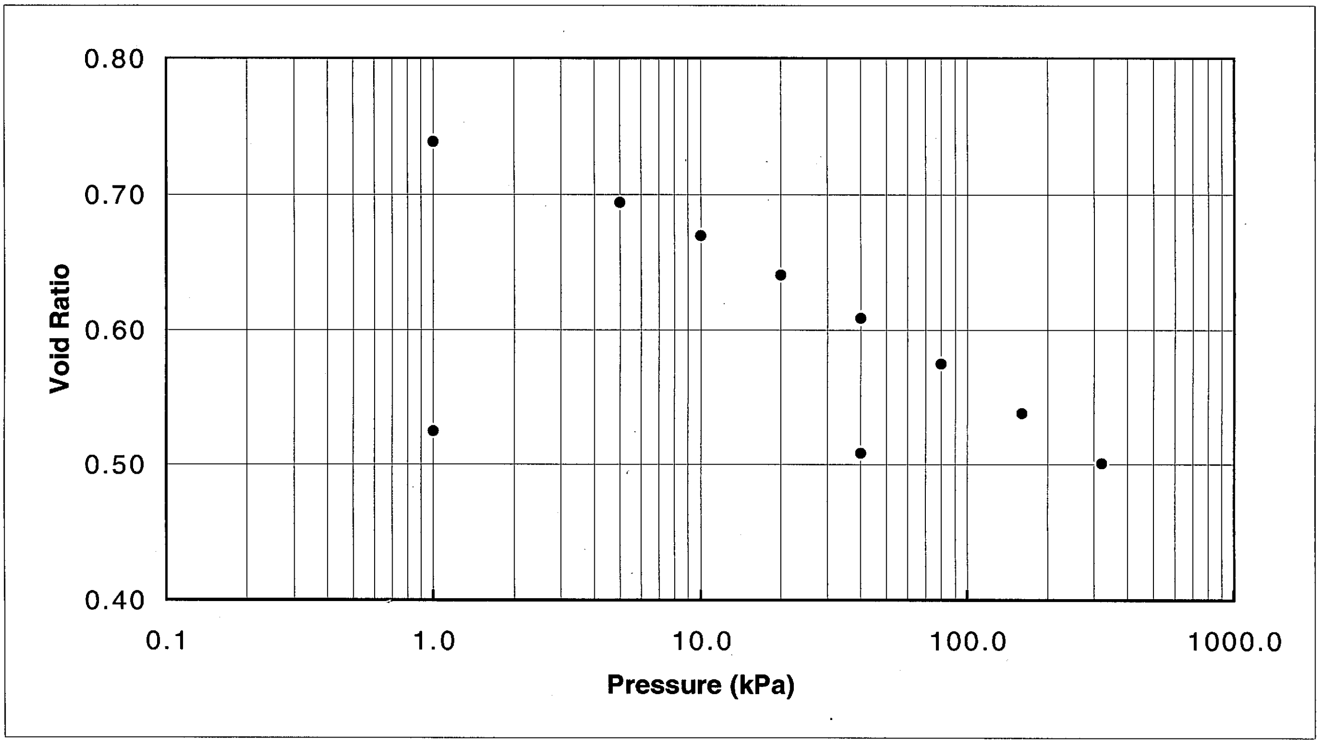
Angle of Shear:

CONSOLIDATION TEST **TAIL LAKE AND JETTY 2006**

Project No.: 1780176
Date Tested: 06-07-18
Test No.: C-5
Sample ID: SRK06-17-03

	Initial	Final
Height (mm):	26.56	23.30
Moisture (%):	27.56	20.03
Wet Dens. (Mg/m3):	1.983	2.128
Dry Dens. (Mg/m3):	1.555	1.773
Void Ratio:	0.7390	0.5252
Saturation:	100	100

Spec.Grav.= 2.70



P (kPa)	Void Ratio	Cv (m2/yr)	Mv (m2/MN)	K (m/s)
1	0.74			
5	0.6945	0.27	6.4031	5.394E-10
10	0.6698	0.30	2.9153	2.726E-10
20	0.6405	0.43	1.7509	2.329E-10
40	0.6086	0.71	0.9722	2.147E-10
80	0.5749	0.88	0.5240	1.436E-10
160	0.5380	1.46	0.2931	1.328E-10
320	0.5012	2.72	0.1495	1.265E-10
40	0.5087		0.0179	
1	0.5252		0.2808	

SAMPLE INFORMATION

Project: Tail Lake and Jetty 2006

Address:

Project Number: 1780176

Date Tested: 06.07.18 By: S.K.

Test Apparatus: Consolidation

Machine Number: 8

Rate of Strain: mm% / minute

Normal Stress: kPa

Cell Pressure: kPa

Back Pressure: kPa

Head Differential: kPa

Swelling Pressure: kPa

Borehole Number: SRK06-17-03

Depth:

Test Number: C-5

Sample Description: CLAY, silty, reconstituted.

Sample Description	Diameter (mm)	Height (mm)
1		
2		
3		
4		
Mean	49.96	26.56

$V = 52.07\ cm^3$

	Trimmings	Initial	Final
Tare Number			
Mass of Wet Soil & Tare g		187.34	(97.18) 187.73
Mass of Dry Soil & Tare g			(80.96) 171.55
Mass of Tare g		84.07	6.70
Mass of Dry Soil g			
Mass of Moisture g			
Moisture Content %		27.56	20.03
Wet Density Mg/m^3		1.983	2.128
Dry Density Mg/m^3		1.555	1.773

6.83
6.65
.18

Sketch and Remarks:

Angle of Shear:

eba

EBA Engineering

Unconsolidated Undrained Triaxial Test **Tail Lake and Jetty 2006**

Project No.: 1780176
Date Tested: 06-07-27

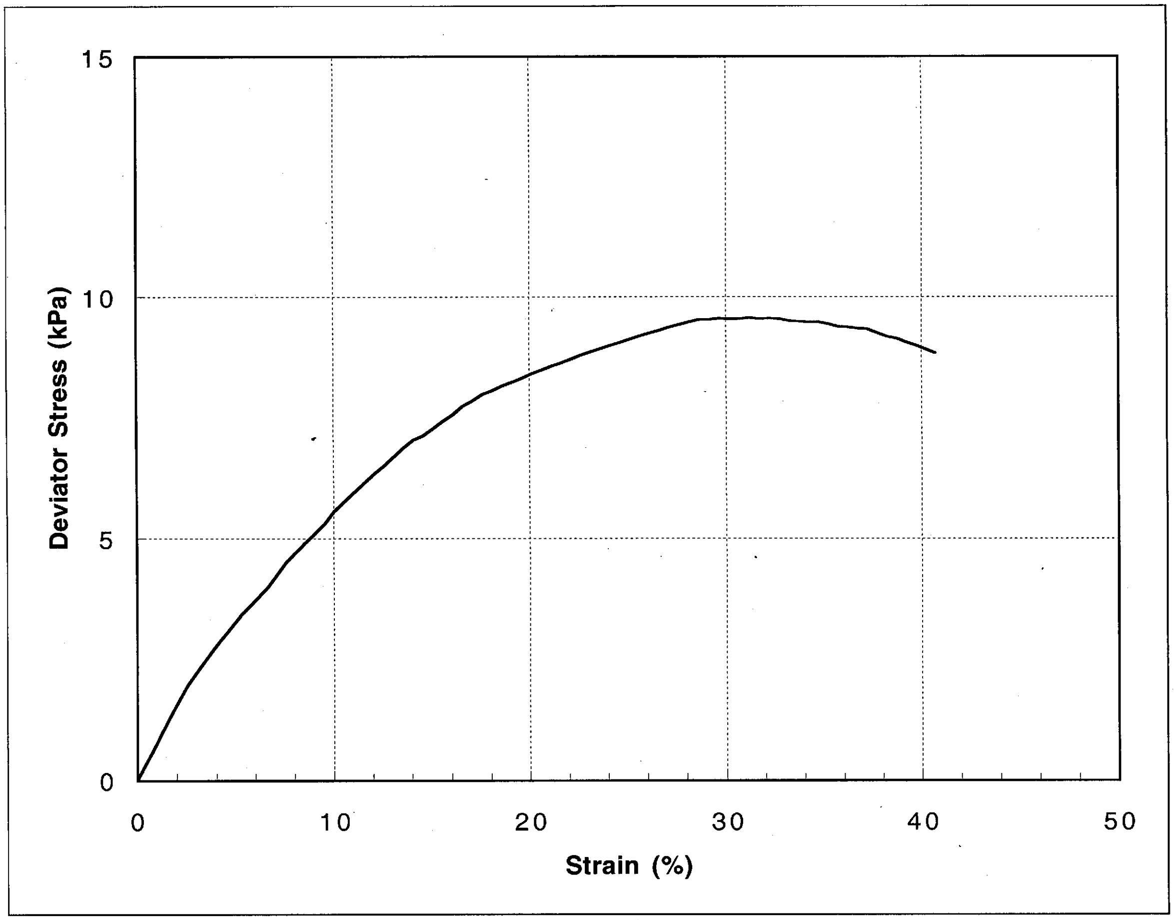
Sample ID: SRK06-12-02
Test Number: UU-3
Confining Stress (kPa): 130

Initial Sample Conditions

Moisture Content (%):	44.3
Wet Density (Mg/m^3):	1.918
Dry Density (Mg/m^3):	1.329

Rate of Strain (%/min.): 0.5

Peak Stress (kPa): 9.6



SAMPLE INFORMATION

Project: Tail Lake and Jetty 2006

Address:

Project Number: 1780176

Date Tested: 06.07.27 By: S.K.

Test Apparatus: TX (UU)

Machine Number: 1

Rate of Strain: 0,5 ~~mm~~% / minute

Normal Stress: kPa

Cell Pressure: 130 kPa

Back Pressure: kPa

Head Differential: kPa

Swelling Pressure: kPa

Borehole Number: SRK06 - 12 - 02

Depth:

Test Number: UU - 3

Sample Description: CLAY, silty, some sand, very soft, wet, dark gray.

Sample Description	Diameter (mm)	Height (mm)
1		
2		
3		
4		
Mean	41,20	74,50

$V = 99.32\ cm^3$

	Trimmings	Initial	Final
Tare Number			
Mass of Wet Soil & Tare g		190.53	191,97
Mass of Dry Soil & Tare g			135,07
Mass of Tare g			6,65
Mass of Dry Soil g			128,42
Mass of Moisture g			
Moisture Content %			44.31
Wet Density Mg/m^3		1.918	
Dry Density Mg/m^3		1.329	

Sketch and Remarks:

Note: Sample slumped somewhat during set up.

Angle of Shear:

Unconsolidated Undrained Triaxial Test

Tail Lake and Jetty 2006

Project No.: 1780176
Date Tested: 06-07-28

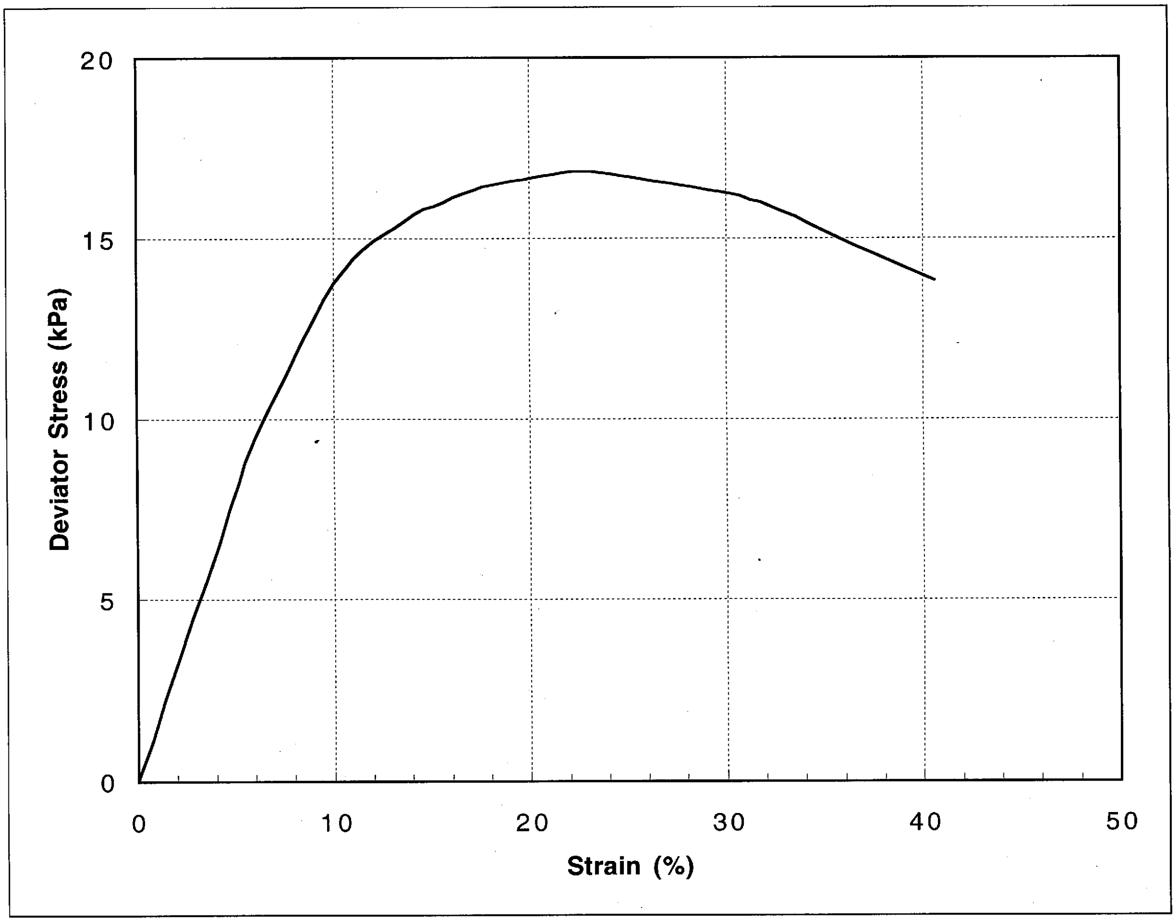
Sample ID: SRK06-16-02
Test Number: UU-4
Confining Stress (kPa): 111

Initial Sample Conditions

Moisture Content (%):	31.8
Wet Density (Mg/m^3):	2.076
Dry Density (Mg/m^3):	1.576

Rate of Strain (%/min.): 0.5

Peak Stress (kPa): 16.8



SAMPLE INFORMATION

Project: Tail Lake and Jetty 2006

Address:

Project Number: 1780176

Date Tested: 06.07.28 By: S.K.

Test Apparatus: TX(UU)

Machine Number: 1

Rate of Strain: 0.5 % / minute

Normal Stress: kPa

Cell Pressure: 111 kPa

Back Pressure: kPa

Head Differential: kPa

Swelling Pressure: kPa

Borehole Number: SRK06-16-02

Depth:

Test Number: UU-4

Sample Description: CLAY, silty, some sand, soft, dark gray

Sample Description	Diameter (mm)	Height (mm)
1		
2		
3		
4		
Mean	38.64	77.40

V = 90.76 cm^3

	Trimmings	Initial	Final
Tare Number			
Mass of Wet Soil & Tare g		188.45	192.12
Mass of Dry Soil & Tare g			147.39
Mass of Tare g			6.65
Mass of Dry Soil g			140.74
Mass of Moisture g			
Moisture Content %			31.78
Wet Density Mg/m^3		2.076	
Dry Density Mg/m^3		1.576	

Sketch and Remarks:

Note: Sampled slumped somewhat during set up.

Angle of Shear:

eba

Appendix C

Technical memorandum outlining jetty design calculations

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vancouver@srk.com
www.srk.com

Tel: **604.681.4196**
Fax: **604.687.5532**

Technical Memorandum

To:	Jim Currie	**Date:**	March 15, 2007
cc:	Project File	**From:**	Maritz Rykaart
Subject:	Jetty Design Calculations	**Project #:**	1CM014.008

1 Introduction

This memo documents bearing capacity design calculations and assumptions for the proposed continuous rock fill jetty in Roberts Bay, Hope Bay, Nunavut, Canada. This jetty will be part of the development infrastructure for the proposed Doris North Project, a gold mine being developed by Miramar Hope Bay Ltd. (MHBL). Background information is documented in the following report;

> SRK Consulting (Canada) Inc. 2005. *Preliminary Jetty Design, Doris North Project, Hope Bay, Nunavut, Canada*. Technical report submitted to Miramar Hope Bay Limited, Project No. 1CM014.006, October 2005.

2 Design

2.1 Design Approach

The continuous rock fill jetty will be constructed on soft marine sediments. It is therefore necessary to confirm that the load applied by the jetty will be less than the allowable (ultimate) bearing capacity of the marine sediments. For the purpose of this design it is reasonable to assume that the base of the rock fill jetty is a shallow foundation (Holtz and Kovacs 1981).

The bearing capacity calculations presented have been completed in accordance with standard bearing capacity theory (Holtz and Kovacs 1981), with design input parameters based on site specific measured properties.

2.2 Data Sources

Geotechnical data for the jetty foundation material have been documented in the Preliminary Jetty Design Report mentioned in Section 1 of this memo. This data includes drill holes, in-situ vane shear testing and laboratory foundation indicator testing. Subsequent to the completion of this report another field program was carried out to characterize the jetty foundation materials (SRK 2006). Together these data sets are deemed adequate to conduct the jetty design in accordance with the adopted design criteria as set out by MHBL.

2.3 Applied Loads

2.3.1 Dead Loads

The limiting case for the proposed jetty geometry is described by a cross section through the jetty head. This jetty head consists of a 25 m wide roadway crown over a 6.5 m deep fill with side slopes of 1.2:1. Under this scenario, the base of the foundation is 40.8 m wide, and the water level is 1.5 m below the roadway (with the maximum water depth being 5 m).

For a 1 m deep section through the jetty head fill, the volumes, unit weights, and total dead load for the geometry described above are included in Table 1.

Table 1. Jetty head section volumes, unit weights, and loads.

Section	Volume m^3	Unit Wt. (kN/m^3)	Load (kN)
Unsaturated upper fill	40.2	19.62	789
Saturated lower fill	173.0	9.81 (submerged)	1,697
Total fill	**213.2**		**2,486**

Thus, for a total area of 40.2 m^2, the applied load of 2,486 kN, due to the weight of the fill, results in an applied stress, q_a, of 61.8 kPa over the area of the footing.

2.3.2 Live Loads

Live loads on the jetty include the traffic of loaders, as well as the action of ice, wind, and snow. The total load applied by a Komastsu WA500-3 Wheeled Loader (the largest equipment to be used) with a fully laden shipping container is approximately 48,100 kg. Over a 1 m deep section of the jetty head, the applied load is equivalent to an additional increase in applied stress, q_a, of 1.2 kPa.

Additional live lateral loads associated with ice, wind, snow and waves, as well as the barge itself have not been considered. The magnitude of these additional loads, and thus their effect on the structure has not been assessed. MHBL believes that the design life of the jetty is short-lived and thus they will rather embark on a maintenance program for the jetty than provide a more robust design at this time. The consequences and impacts of this on the structure and surrounding environment have been discussed in the main text to which this memo has been attached as a memo. MHBL has been made aware of the risk of this approach, and has opted to accept it.

2.3.3 Total Load

The total load exerted by the jetty on the marine foundation is thus the sum of the live and the dead load, i.e. 61.8 + 1.2 = 63 kPa.

2.4 Bearing Capacity

Nilcon vane shear test results for the upper 5 m of marine sediment at the jetty head location are summarized in Table 2.

Table 2. Nilcon vane shear test results for proposed jetty head location.

	Peak (kPa)	Residual (kPa)	Remoulded (kPa)
Maximum	28.1	12.7	4.4
Minimum	13.3	4.7	0.6
Average	20.4	8.6	3.2

The bearing capacity of the sediment was calculated on the basis of peak undrained shear strength of 15 kPa. The average plasticity index of CL samples taken from the proposed jetty location, and from the 1997 investigation in the area (EBA 1997) was 17.5%, and no vane shear correction was applied to field values. The choice of these values are supported by the 2006 characterization data (SRK 2006) which showed measured peak undrained shear strength between 10 and 16 kPa.

For undrained loading at the surface of the marine sediment, the ultimate bearing capacity equation reduces to:

$q_u = N_c C_u$

where, q_u is ultimate bearing capacity, N_c is a bearing capacity coefficient, and C_u is the undrained shear strength. The value of N_c for a soft sediment varies to a maximum of 5.14. Accordingly, the ultimate bearing capacity, q_u, of the sediment is 77.1 kPa.

2.5 Bearing Capacity Factor of Safety

The factor of safety is calculated as follows:

F.S. = q_u/q_a = 77.1 kPa / 63 kPa = 1.22

Depending on the type of structure an acceptable factor of safety against bearing capacity failure would be between 1.5 and 3. Acceptance of a factor of safety of 1.22 for the jetty is a risk that MHBL has been made aware of and they accept the consequences thereof.

It should be noted that SRK recommends that MHBL construct the jetty on two layers of geogrid, which will result in an undetermined increase in the factor of safety (see later). Also, as shown later, by flattening the slope angles of the jetty, the factor of safety can be increased.

2.6 Consolidation Settlement

2.6.1 Total Settlement

The proposed jetty will undergo settlement due to the consolidation of the underlying marine sediment. Consolidation testing was carried out and the results are reported in SRK (2006). The total settlement and time to consolidation presented in this memo are based on sample void content as determined from saturated water content, the depth of the sediment layer, and assumed values of compression index and coefficient of consolidation. Values of parameters used for the calculation of total settlement are included in Table 3.

Table 3. Design values for consolidation calculations.

Component	Value
Thickness of marine sediment layer	13 m
Saturated unit weight of marine sediment	18 kN/m^3
Initial effective stress at midpoint of the layer	53.2 kPa
Initial void ratio	1.27
Compression Index	0.25 to 0.5 (assumed)
Applied stress	62 kPa
Coefficient of consolidation	10 m^2/year (assumed)

Assuming an increase in effective stress equal to the dead load of 61.8 kPa, the midpoint of the profile will undergo a change in effective stress from 53.2 kPa to 115.2 kPa. The total expected settlement is estimated to be approximately 0.5 m to 1.0 m.

2.6.2 Time Rate of Consolidation

Estimates of time of consolidation indicate up to 0.15 m settlement after one year, and up to 0.3 m after 5 years. The actual rates of settlement may vary considerably from estimates.

Rates of consolidation are estimated from coefficient of consolidation. The coefficient of consolidation of 10 m^2/year listed in Table 3 was approximated from the average liquid limit of near 40% and Figure 9.10, page 404, Holtz and Kovacs (1981).

Time to consolidation is highly dependent on the hydraulic conductivity of the sediment, which was not measured. The drilling program observed some sandier sediments, which will have higher hydraulic conductivity than clay rich portions. The presence of sandy layers may increase the rate of consolidation.

3 Design Options

Alternative geometries and the effect of including geosynthetic re-enforcement of the base of the jetty fill were examined for effect on applied load q_a and factor of safety, F.S.

3.1 Jetty Head Geometry

Options for decreasing the pressure at the base of the jetty head fill include flattening the side slopes, and reducing the width of the fill. The variation in applied stress is illustrated in Table 4. The most conservative design includes a design profile with a 6 m roadway with 4:1 side slopes. Predicted loads are converted to factors of safety, F.S.'s, in Table 5.

Table 4. Variation in applied stress, q_a, due to changes in jetty head geometry.

Side Slope (H:V)	Top Width (m)			
	25	15	10	6
1.2:1	61.2	55.6	51.1	46.1
2:1	55.6	50.0	46.1	42.0
3:1	51.1	46.1	42.7	39.6
4:1	48.2	43.6	40.8	38.2

Table 5. Factor of Safety for alternate jetty head geometries (excluding live loads).

Side Slope (H:V)	Top Width (m)			
	25	15	10	6
1.2:1	1.26	1.39	1.51	1.67
2:1	1.39	1.54	1.67	1.84
3:1	1.51	1.67	1.80	1.95
4:1	1.60	1.77	1.89	2.02

3.2 Geosynthetic Re-Enforcement

The use of geosynthetic (geotextile and geogrid) re-enforcement at the base of the fill was investigated for effect on bearing capacity (Koerner 2005). Two suppliers were also contacted for information regarding the use of geosynthetics. Principle advantages to using a geosynthetic re-enforcement at the base of the jetty fill include:

- Prevent rock fill from sinking upon initial placement during construction
- Reduction of differential settlements
- Even distribution of stress over marine sediment – allowing use of N_c = 5.14
- Prevent movement of fines into overlying coarse layers

The soft marine sediments at the proposed jetty location may fail during construction if the ultimate bearing capacity is exceeded. With time, the sediments will consolidate, and the allowable load will increase. However, localized loading may cause a failure, and a geosynthetic re-enforced pad will help reduce the potential for failure.

Dwg. J-01, J-02 and J-03 of Supporting Document S-04 provide details of the final jetty design which includes two layers of geogrid. The configuration and type of geogrid was recommended by a supplier and is not based on a design carried out by SRK. Detailed specifications of the recommended geogrid are provided in Supporting Document S-03.

Case studies where geosynthetics has been used for this type of application are listed on the web site of one supplier (www.tenax.net/geosynthetics/case_history). SRK is not aware of any case study of geosynthetic re-enforced pad constructed in an arctic environment. However, geosynthetics are commonly used in conventional applications in the arctic (liners, ponds, etc.), and therefore there is no reason to believe that this application would not be feasible. This statement is supported by the suppliers that were contacted.

4 References

EBA Engineering Consultants. 1997. *Boston Gold Project Geotechnical Investigation Proposed Roberts Bay Port*. Report Submitted to BHP World Minerals, October.

Holtz, R.D., and Kovacs, W.D. 1981. *An Introduction to Geotechnical Engineering*. Prentice-Hall Inc. New Jersey, pp.733.

Koerner, R.M. 2005. *Designing with Geosynthetics, Fifth Edition*, Pearson Prentice Hall, N.J., 796 pages.

SRK Consulting (2006). *Phase III Foundation Investigation Proposed Roberts Bay Jetty Location, Doris North Project, Nunavut, Canada*. Report prepared for MHBL, August 2006.

Appendix D

Technical memorandum on thermal pad design

SRK Consulting
Engineers and Scientists

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Tel: 604.681.4196
Fax: 604.687.5532

Technical Memorandum

To:	Brian Labadie	**Date:**	August 20, 2006
cc:	Project File	**From:**	Michel Noël/Maritz Rykaart
Subject:	Doris North Project - Thermal modelling to support design thickness for granular pads	**Project #:**	1CM014.008.420

1 Introduction

This technical memorandum presents the modelling that was carried out to determine the minimum thickness for granular pads used for surface infrastructure foundations at the Doris North Project. The granular fill material will consist of crushed rock, and will be placed directly onto the sensitive permafrost overburden soils during the winter.

The intent is to carry out no excavation of permafrost soils, except in areas where there is only a very shallow veneer of overburden covering competent bedrock, and then only for the most critical structures, i.e. the crusher and mill foundations, as well as the fuel tank farm and the airstrip. Therefore, all other surface infrastructure must be constructed on rockfill pads that will remain stable, but that will not result in undue damage to the permafrost environment. It is furthermore understood that upon closure, the pads will not be completely removed, thus it is not expected that the site be returned to its pre-mining state.

The thickness of the granular pads will be dependent on the required bearing capacity and on the thermal behaviour in relation with the permafrost. The thickness of the granular pads discussed herein was calculated using the modified Berggren equation developed by Aldrich and Paynter (1966).

2 Input Parameters

The granular pads will be fabricated from crushed basalt rock. The thermal properties were estimated using the method by Johansen (1975) and had the following properties:

- Porosity: 0.30
- Degrees of saturation: 60%
- Unsaturated thermal conductivity: unfrozen: 161 kJ m^{-1} day^{-1} $°C^{-1}$
  frozen: 178 kJ m^{-1} day^{-1} $°C^{-1}$
- Unsaturated volumetric heat capacity: unfrozen: 2,230 kJ m^{-3} °C
  frozen: 1,916 kJ m^{-3} °C

Climatic data was collected at the Doris North and the Boston Camp sites during exploration work. But because of limited data, the local climatic data was complemented using three regional weather stations operated by Environment Canada, namely Lupin, Ikaluktutiak (Cambridge Bay) and Kugluktuk (Coppermine) (AMEC 2003a, b). The climatic data collected at the Doris North and Boston Camp sites was then used to develop correlations for the Doris North site using the Environment Canada weather stations.

The correlated data from the Environment Canada weather stations over a 30 year period give the following values:

- mean annual ambient temperature: -12.1 °C
- amplitude of annual ambient temperature: 20.3 °C
- air thawing index: 748 °C-days
- air freezing index: -5,135 °C-days
- days with mean daily temperature above freezing: 108

The surface temperature was assigned a value of -6 °C. The ground temperature measured at the site outside the influence of water bodies averaged about -8 °C over a range of -10 to -6 °C (SRK 2005a, b).

3 Results

Using the method by Aldrich and Paynter (1966) with the input values listed herein, a pad thickness of about 2.1 m would be required to maintain the active zone within the granular pad, i.e. the original ground is below the active zone and remains permanently frozen. This recommended pad thickness can be reduced if the original ground does not contain massive ice within the active zone while having good draining capabilities (i.e. sand deposits). In this case, the thickness of the granular pad would then be controlled by bearing capacity requirements. On bedrock outcrops; the pad thickness would be determined by the grading requirements.

It should further be noted that the Doris North site ground surface is generally covered with hummocky vegetation or by muskeg where overburden is present. Such organic layer provides good insulation to the underlying permafrost but is sensitive to disturbance. The removal of the organic layer will increase the depth of the active layer. Basic thermal simulations indicate that the thermal value of the organic cover can be approximated by about 1 m of granular fill, i.e. if the organic layer was to be removed, it should be replaced by at least 1 m of granular fill to ensure that the active layer remains unchanged.

4 Design Recommendations

If the pad thickness is not sufficiently thick to ensure that the active layer remains within the pad fill material, then the depth to which the active layer does penetrate beneath the pad will consolidate when the soil thaws, which may lead to settlement and subsequent damage to the foundation pad and any associated infrastructure on the pad.

The extensive geotechnical investigations carried out at the Doris North site does confirm that the overburden soils are ice rich; however, these ice rich zones are generally not found within the active layer which ranges between 0.5 to 2 m thick. Therefore, having absolute design criteria that requires the active layer to remain within the construction pad is probably not necessary, since settlement is likely to be small. Furthermore such settlement is not likely to occur rapidly, but could take days, or more likely weeks and months to produce noticeable results. Such improvements could thus easily be managed and mitigated through the adoption of a regular monitoring and maintenance program.

Monitoring should include installation of thermistor cables to determine how deep the active layer penetrates beneath the pad, as well as visual observation of pads and road alignments. Mitigation will consist of a program of infill and levelling of pad and roadway surfaces using pre-stockpiled and graded fill material.

For the preliminary design stage of the Doris North Project, SRK recommended that MHBL adopt a minimum pad thickness of 2.5 m for structures that would be susceptible to damage from settlement, such as the mill and crusher foundations, and for less important structures such as roads, a pad thickness of 2 m would be sufficient. This decision was made at the time with a limited understanding of the physical site conditions and therefore the highest margin of conservatism was adopted. Furthermore, MHBL did not wish to underestimate the potential costs associated with capital construction for the Project.

For the final detailed design stage, MHBL requested that SRK consider reducing the pad thickness requirement, taking into account the additional information that is available about the site physical conditions. Reducing the pad thickness requirement would not only result in a significantly lower amount of quarry development and thus a lower environmental impact, but could also offer some cost saving to the Project.

SRK would be satisfied that all non-critical pads be have a minimum overall thickness of 1.0 m. This thickness will ensure physical stability based on the expected loads, and also in some areas it will be sufficiently thick that the active layer will remain within the pad. In those areas that the active layer will extend beneath the pad, MHBL is advised that settlement will occur, and that such settlement will lead to the need to be monitored and repairs will have to be carried out to ensure safe and efficient operation. MHBL is also advised that in some instances settlement may lead to the temporary closure of roads or facilities until the necessary repairs have been completed.

For important structures, the minimum pad thickness should be 2.0 m. whilst this is probably sufficiently thick that the active layer would remain in the fill material, there does remain a small possibility for some settlement, so MHBL should put in place a monitoring and maintenance plan as described previously that includes these structures.

This reduced pad thickness will not result in any greater environmental impact on the permafrost environment, especially since the fill will not be removed at closure.

5 REFERENCES

AMEC. 2003a. *Draft Environmental Impact Statement, Doris North Project, Nunavut, Canada*. Report submitted to Miramar Hope Bay Limited, January 2003.

AMEC. 2003b. *Meteorology and Hydrology Baseline, Doris North Project, Nunavut, Canada*. Report submitted to Miramar Hope Bay Limited, August 2003.

Andersland, O.B. and Ladanyi, B. 1994. *An introduction to frozen ground engineering*. Chapman & Hall Inc., 352 pages.

Aldrich, H.P. and Paynter, H.M. 1966. *Depth of Frost Penetration in Non-uniform Soil*. U.S. Army Cold Reg. Res. Eng. Lab. Spec. Rep. 104.

Johansen, O. 1975. *Thermal conductivity of soils*. Ph.D. diss., Norwegian Technical Univ., Trondheim; also, U.S. Army Cold Reg. Res. Eng. Lab. Transl. 637, July 1977.

SRK Consulting (Canada) Inc. 2005a. Preliminary Surface Infrastructure Design, Doris North Project, Hope Bay, Nunavut, Canada. Report submitted to MHBL, Project No. 1CM014.006, October.

SRK Consulting (Canada) Inc. 2005b. Preliminary Tailings Dam Design, Doris North Project, Hope Bay, Nunavut, Canada. Report submitted to MHBL, Project No. 1CM014.006, October.

Appendix E

Technical memorandum on construction quantities

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Technical Memorandum

To:	Larry Connell	**Date:**	October 25, 2006
cc:	Project File	**From:**	Maritz Rykaart
Subject:	Construction Quantities for the Doris North Project	**Project #:**	1CM014.008

All surface infrastructure components for the Doris North Project that require fill, i.e. roads, runway, pads etc. will be constructed from clean rock quarried from one of the four quarry sites identified.

Table 1 below lists preliminary volumes of construction rock, as well as the most likely quarry source. These volumes and footprint areas are based on the Preliminary designs carried out in October 2005. Revised quantities will be calculated as Schedule 1 to the Technical Specifications at the time of tender.

The construction rock properties have been assumed to be as follows;

- specific gravity = 2.50,
- swell = 40%,
- load cubic metre (LCM) density = 1.79 Mg/m^3,
- moisture content = 4%,
- reconsolidation = 50%, and
- reconsolidated (excavated cubic metre (ECM)) density = 2.08 Mg/m^3.

Table 2 lists estimated quarry rock requirements for maintenance of the surface infrastructure components. Additional construction rock volumes that may be required at mine closure is listed in Table 3. Finally, Table 4 list a cumulative total quarry rock volume expected to be developed from each of the four rock quarries.

Table 1: Estimated Footprint Size, Quarry Rock Volumes (Neat) and Quarry Locations for Surface Infrastructure Components

Infrastructure Component	General Detail	Estimated Quantity[1]		Footprint Surface[1] Area (m^2)	Quarry Source
		ECM (m^3)	Dry Tonnes		
Jetty	6m wide traffic surface; 1.2:1 side slopes; 0.5m sediment consolidation; 103m length (SRK 2005b)	5,600	11,600	1,800	Q1
Jetty (contingency)	Allowance for excessive slumping and settlement during construction (SRK 2005b)	2,800	5,800	900	Q1
Beach lay-down area	60m x 100m surface area; 1.2:1 side slopes; 2.5m average thickness	16,300	33,800	6,700	Q1
Fuel transfer station	32m x 16.5m surface area; 1.2:1 side slopes, 3m average thickness; 0.8m high containment berm	2,000 (300 m^2 HDPE, 600 m^2 geotextile)	4,000	600	Q1
Tank farm at mill (7.5 million litre)	71m x 71m surface area; 1.2:1 side slopes; 0.5m average thickness; 0.8m high containment berm	5,200 (4,700 m^2 HDPE, 9,400 m^2 geotextile)	10,800	5,000	Q2
Tailings discharge decant road	5.1m wide traffic surface; 1.2:1 side slopes; 2.0m average thickness; 378m length	5,700	11,900	2,400	Q2
Tailings discharge pump house pad	20m x 20m surface area; 1.2:1 side slopes; 2.0m average thickness	700	1,500	400	Q2
All-weather road (barge site to mill)	6m wide traffic surface; 1.2:1 side slopes; 2.0m average thickness; 4.8km length	80,700	167,800	51,900	Q1 (20%) Q2 (80%)
Road turnouts (2) (barge site to mill)	10m wide; 30m long; 1.2:1 side slopes; 2.0m average thickness	1,200	2,500	800	Q1
All-weather road (tailings service road)	5.1m wide traffic surface; 1.2:1 side slopes; 2.0m average thickness; 5.9km length	88,500	184,100	59,000	Q2
Caribou crossings (8)	10m long; 5:1 approach slopes; 2.0m average thickness	2,500	5,200	2,500	Q2
Road turnouts (8) & turnaround (tailings service road)	10m wide; 30m long; 1.2:1 side slopes; 2.0m average thickness & 10m x 10m turnaround	5,000	10,500	3,100	Q2
Explosives magazine access road	5.1 m wide traffic surface; 1.2:1 side slopes; 2.0m average thickness; 525m length	7,900	16,400	5,200	Q2
Float plane & boat dock service road	6m wide traffic surface; 1.2:1 side slopes; 2.0m average thickness; 300m length	8,500	17,600	3,300	Q2
Landfill access road	6m wide traffic surface; 1.2:1 side slopes; 2.0m average thickness; 150m length	2,600	5,300	1,600	Q2

Infrastructure Component	General Detail	Estimated Quantity[1]		Footprint Surface[1] Area (m^2)	Quarry Source
		ECM (m^3)	Dry Tonnes		
Bridge crossing and abutments (2)	10m wide traffic surface; 1.2:1 side slopes; 2.5m average thickness; 27m length	1,900	3,800	900	Q2
Permanent all-weather airstrip	23m wide traffic surface; 2.5:1 side slopes; 2.5 m average thickness; 914m length	66,900	139,100	32,500	Q2
Airstrip apron	17m x 40m surface area; 2.5:1 side slopes; 2.5m average thickness	2,000	4,100	1,600	Q2
Explosives magazines	3 pads, total 550m^2 surface area; 1.2:1 side slopes; 2.5m average thickness; safety berm; AN/FO pad	8,500	17,600	1,700	Q2
Mill and camp area	Mill Crusher Ore Stockpile Workshop Fuel tank farm Mill reagents storage Lay-down area Power supply Camp/Dry Mine office Sewage treatment plant Potable water treatment plant Waste rock pile pad and berm Waste rock pile pond berm	55,100 (1,000m^2 HDPE, 2,000m^2, geotextile)	114,600	62,600	Q4
Float plane & dock	10m x 30m surface area; 1.2:1 side slopes; 3.0m average thickness	900	1,900	1,000	Q2
Tailings emergency dump catch basins (4)	25.2m x 25.2m surface area; 2:1 side slopes; 2.0m average base thickness; 1m high containment berm	5,100 (1,200 m^2 HDPE, 1,200 m^2 geotextile)	10,600	4,400	Q2
North Dam	Refer to SRK (2005a) for details of this structure	65,400	136,100	12,100	Q2
South Dam	Refer to SRK (2005a) for details of this structure	42,100	87,400	12,800	Q2
Roberts Bay fish habitat	8 spurs with each 5m x 15m surface area; 0.5m thickness; and 6 rock spurs each with 5m x 20m surface area; 0.5m thickness (Golder 2005)	600	1,200	1,200	Q1
Doris Lake fish habitat	5 areas each with 25m x 25m surface area; 1.5m thickness; and 1 area with 30m x 30m surface area; 1.5m thickness (Golder 2005)	6,000	12,500	4,000	Q2
Shoreline protection	20% of 12.9 ha surface area (up to elev. 29.4m); 0.5m thickness (SRK 2005c)	12,900 (25,800m^2 geotextile)	26,800	25,800	Q3
TOTALS		**502,600**	**1,044,500**	**305,800**	**-**

1. All estimated quantities and areas have been rounded to nearest 100.

Table 2: Estimated Quarry Rock Volumes Required for Maintenance of the Surface Infrastructure Components

Infrastructure Component	General Detail	Estimated Quantity[1]		Footprint Surface[1] Area (m^2)	Quarry Source
		ECM (m^3)	Dry Tonnes		
Jetty maintenance	Allow 50cm to be added to Jetty surface every year for 5 years	1,400	2,800	n/a	Q1
All surface road maintenance	Allowance for all surface road maintenance @ 10cm new surfacing grade every year for 8 years	73,000	151,800	n/a	Q1 (10%) Q2 (90%)
Landfill interim cover	100m x 100m surface area; 1.2:1 side slopes; 0.3m average thickness added on top of waste every year for 8 years	24,000	50,000	n/a	Q2
Shoreline erosion (contingency)	20% of 12.9 ha surface area (up to elev. 29.4m); 0.5m thickness (SRK 2005c)	12,900 (25,800m^2 geotextile)	26,800	25,800	Q3
TOTALS		**111,300**	**231,400**	**25,800**	**-**

1. All estimated quantities and areas have been rounded to nearest 100.

Table 3: Estimated Quarry Rock Volumes Required for Closure of Surface Infrastructure Components

Infrastructure Component	General Detail	Estimated Quantity[1]		Footprint Surface[1] Area (m^2)	Quarry Source
		ECM (m^3)	Dry Tonnes		
Landfill closure	100m x 100m surface area; 1.2:1 side slopes; 1m average thickness for ultimate cover	10,000	20,800	n/a	Q2
Shoreline erosion (contingency)	Remaining 60% of 12.9 ha surface area (up to elev. 29.4m); 0.5m thickness (SRK 2005c)	38,700 (77,400m^2 geotextile)	80,500	77,400	Q3
Shoreline erosion (worse case contingency)	36.7 ha surface area (up to full supply level); 0.5m thickness (SRK 2005c)	183,500 (367,000m^2 geotextile)	381,600	367,000	Q3
TOTALS		**232,200**	**482,900**	**444,400**	**-**

1. All estimated quantities and areas have been rounded to nearest 100.

Table 4: Total Volume of Material Excavated from Each Quarry

Quarry	Estimated Quantity	
	ECM (m^3)	Dry Tonnes
#1	52,000	108,000
#2	491,000	1,020,500
#3	248,000	515,700
#4	55,100	114,600
TOTAL	**846,100**	**1,758,800**

Appendix F

Technical memorandum on bridge design calculations

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Memorandum

To:	Maritz Rykaart	**Date:**	January 29, 2007
cc:	Project File	**From:**	Alvin Tong
Subject:	Doris North Project – Doris Creek Bridge Abutment Stability Analysis	**Project #:**	1CM014.008

1 Introduction

A modular steel girder bridge will be constructed on structural fill on the permafrost banks of Doris Creek to provide a permanent free-span crossing for the Doris North Project. Stability analysis was completed to confirm that the abutments would hold up against static and live loads to which the bridge will be subjected. Dynamic loads (earthquakes) are not included in this analysis since the area is not seismically active. This memo documents the methodology, results and recommendations from this analysis.

Abutment settlement calculations have not been carried out since the abutments have been designed to ensure that the permafrost remains intact, i.e. the active layer will move up into the abutment and as such no settlement is expected. Creep settlement is expected to occur; however, this process will take a long time, and will likely not be significant during the life of the bridge.

2 Methodology

The analysis was done with the SLOPE-W software distributed by Geo-Slope International. The outcome is a factor of safety (FOS) analysis on failure planes in a typical cross-sectional profile of the abutment. For each analysis a number of slip circles and centres are evaluated, and the FOS for each possible failure slip plane is calculated.

For this analysis, the surfacing and subgrade materials are not considered because they are not part of structural components of the abutment. Only Run-of-Quarry (ROQ) material and original frozen ground (i.e. permafrost) were considered in the analysis. The boundary limits are extended 10 m from the crest and the toe of the abutment to ensure buffer space of potential slip planes. The water table and pore pressure from the creek flow is not considered since geotechnical investigations (drill holes with thermistors on the abutment locations) has confirmed the presence of permafrost (see main body of the report to which this memo is attached as an appendix). The influence of the stream and its talik is localized. Without the affects of the water table and pore pressures, the permafrost is considered as one uniform stratum. The ROQ material is considered as free draining material and internal pore water pressure will not be encountered. The parameters of the analysis are listed below, and have been based on that used in SD-01 for analysis of the North and South Dams:

- Material Properties:
    - Fill Material bulk density: 20 kN/m^3, ϕ = 40°
    - Frozen Ground: 18.5 kN/m^3, ϕ = 30°
- Bridge Dimensions: 7.5 m wide, 2 m thick, and 30 m long
- Load: Static load = 226,796 kg, Live load = 75,000 kg
- Bridge concrete sill footing = 7.5 m long, 1 m or 1.5 m wide used in the analysis

The static and live load were divided evenly over the concrete sill on the constructed abutment. The loading stress is calculated on the area of the sill.

3 Results

An initial abutment was analyzed, as illustrated in Figure 1. For this configuration the FOS is less than 1.5. Using these dimensions as the baseline, additional sensitivity runs were carried out to determine an optimized configuration that would yield a FOS greater than 2. The variables tested in the sensitivity analysis were:

- Width of the shelf that the sill sits on (vary from 2 m to 3 m)
- Slope of the abutment (vary from 1:1.2 to 1:2)
- Height of the abutment (vary from 4 m to 2 m)
- Width of the footing (vary from 1 m to 1.5 m)
- Embedment of the sill (vary from 0 m to 1 m)

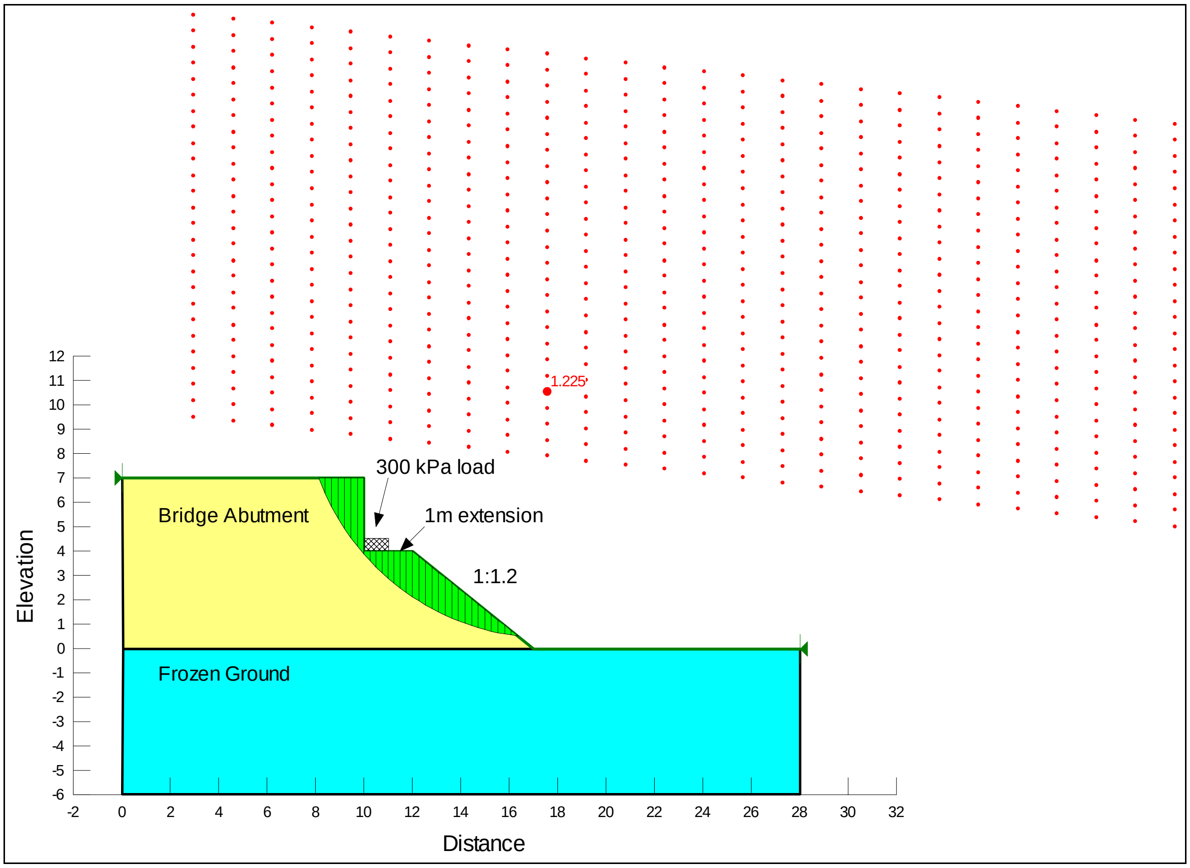

Figure 1: Baseline abutment design

A grid of 625 centres and a band of 12 radii were evaluated to cover all possible slip planes. Different combinations of the variables were analyzed. In some cases deep failure through frozen ground occur at a FOS of less than 2. This scenario is however improbable since the permafrost will remain intact through thermal protection, this acting as a hard rock foundation. The combined loading from the bridge, resulting from the live and dead loads vary between 300 kPa and 132 kPa, which is substantially less that what would normally considered to be a hard rock foundation (600 kPa).

The optimized abutment configuration, which has a FOS greater than 2 are listed below and illustrated in Figure 2.

- Width of the shelf that the sill sits on = 3 m
- The slope of the abutment = 1:2
- The height of the abutment = 2 m
- The width of the footing = 1.5 m
- The embedment of the sill = 1 m

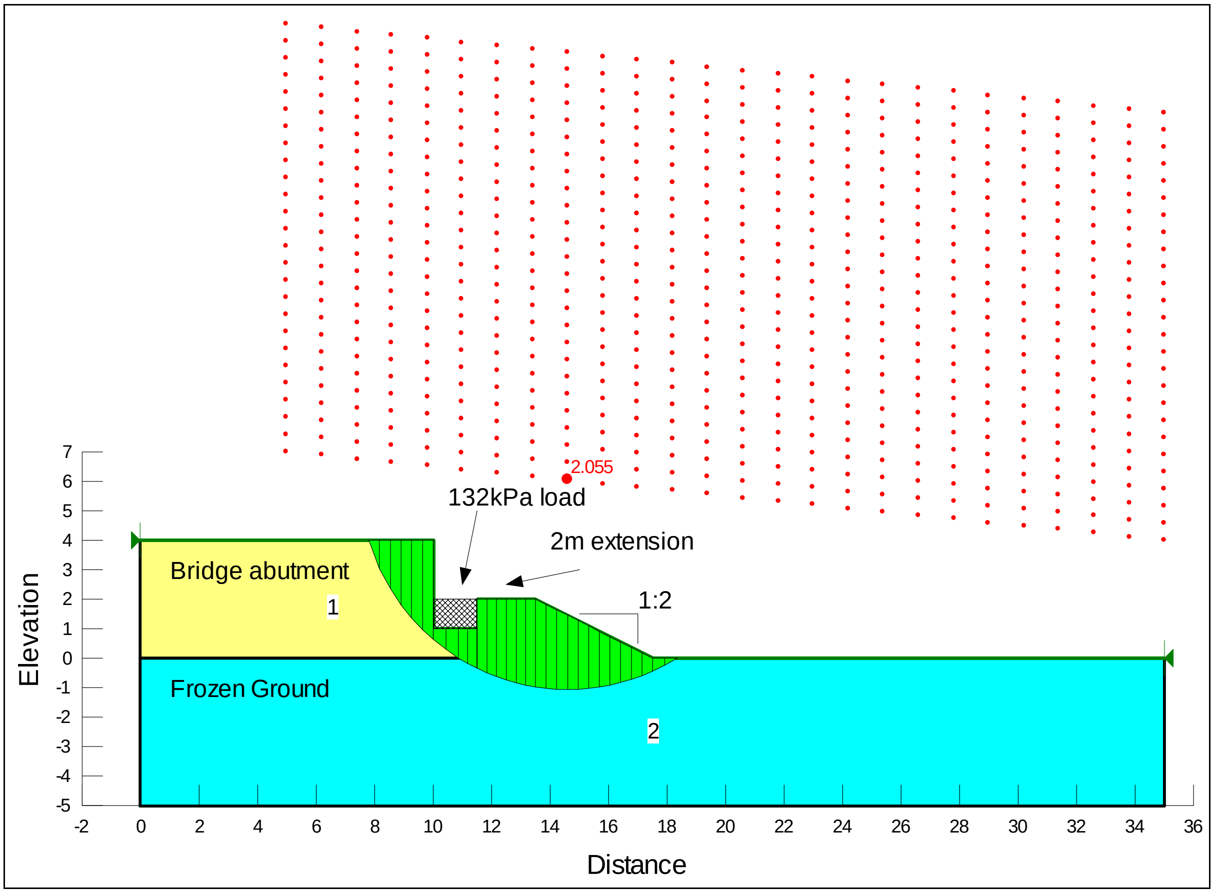

Figure 2: Optimized abutment

3.1 Recommendations

Based on the results of the stability analysis, the bridge abutments have been designed in accordance with the dimensions stipulated in Figure 2.

Appendix G

Hydrotechnical assessment of proposed Doris Creek bridge (Golder Associates)

MEMORANDUM

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Golder Associates Ltd.
Telephone No.: 780-483-3499
Fax No.: 780-483-1574

DATE: 19 January 2007 Proj No. 06-1373-026

TO: Maritz Rykaart, P.Eng.
SRK Consulting

FROM: Nathan Schmidt, Bernard Trevor

RE: Miramar Doris North
Hydrotechnical assessment of proposed Doris Creek bridge

Introduction

Miramar Hope Bay Ltd. (MHBL) proposes to build a bridge at the Doris North project (the Project), across Doris Creek at a location approximately 50 m downstream of the Doris Lake outlet. The proposed plan and elevation views were provided in Drawing S-12, dated October 2006, in the draft water license application to the Nunavut Water Board.

This memorandum describes surveys, hydrological analysis and hydraulic modelling used in the hydrotechnical assessment of the bridge crossing. The results of the assessment include design high water levels, associated mean channel velocities and recommendations for channel erosion protection.

Survey Data

On 2 September 2005, SRK provided coordinates of the left and right abutments of the proposed bridge, as provided in Table 1.

Table 1 – Location of the proposed bridge (UTM NAD83, Zone 13)

Location	Easting (m)	Northing (m)
Left downstream bank (west)	434 037	7 559 479
Right downstream bank (east)	434 077	7 559 470

Table prepared by: NPS *Table checked by:* GRA

On 5 September 2005, topographic surveys at the crossing location were performed by MHBL personnel. These surveys identified the water's edge, bankfull elevation, general land elevations and observed historical high water marks. These were combined with detailed bathymetry measured during a discharge measurements on 11 August 2006 to provide a complete representative cross-section at the bridge location.

The surveys indicated a bankfull elevation at the crossing of 22.0 m, and an observed historical high water elevation of 22.2 m.

Preliminary Waterway Opening Design

The proposed waterway opening at the bridge is shown in Figure 1. The channel at the crossing has a bed elevation of 21.16 m, maximum bankfull depth of 0.84 m and a bankfull width of 13.5 m. The proposed bridge opening allows for a 4.1 m vertical clearance from the bankfull elevation to the lower stringer of the bridge structure. Based on a lower stringer clear span of 26 m and 1.2H:1V headslopes, this provides approximately 4 m of clearance from the top of bank to the toe of each headslope.

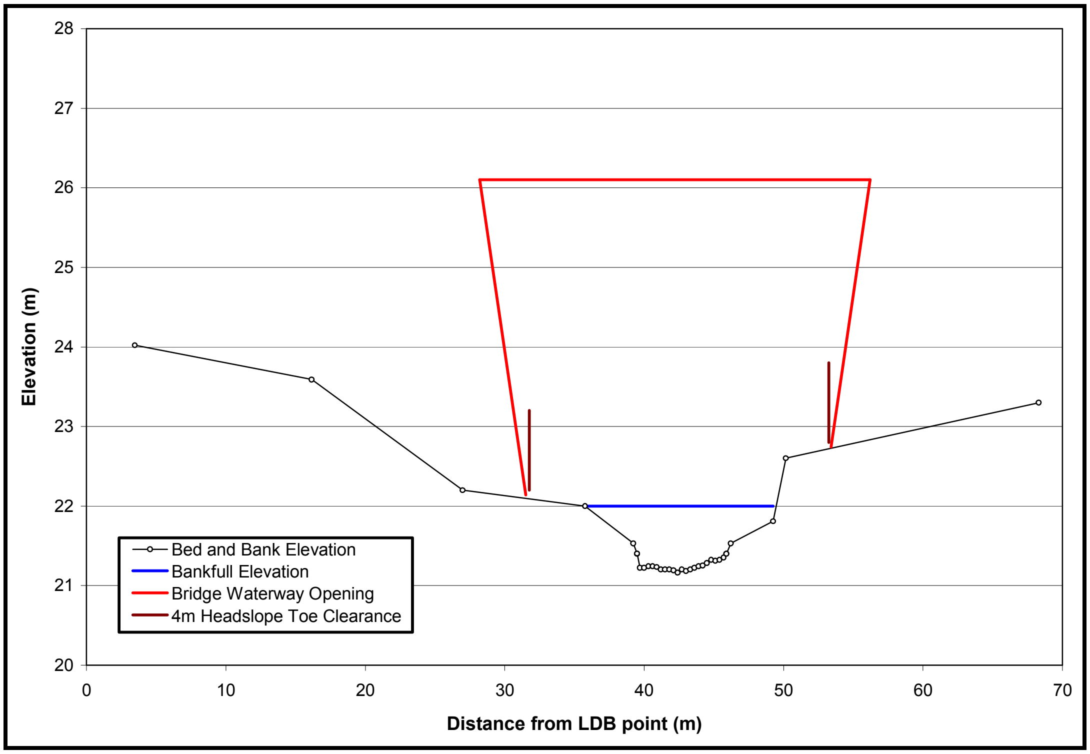

Figure 1 – Proposed waterway opening at Doris Creek bridge

Hydrology

Only limited hydrological monitoring data are available for Doris Creek, and regional data are similarly sparse. Flood peak discharges are expected to be low, because the creek is located:

- at the lower end of a cascade of lakes, where flood peaks will be attenuated;
- in a watershed where the lake surface area is equal to approximately 20% of the watershed area, leading to more attenuation potential; and
- in a watershed where snowmelt floods are greater than those due to rainfall, snowpacks are small and snowmelt is unlikely to be rapid.

Design flood estimates were developed by two methods, described in detail in Appendix I, and summarized below:

1) A peak runoff relationship was developed based on observed relationships between spring snow water equivalent depth and spring snowmelt peak discharge. The derived 1:100 year snowpack was then used to estimate the corresponding peak discharge.
2) High water mark elevations at Doris Lake were used, in conjunction with the established stage-discharge rating curve at the lake outlet, to estimate the discharges corresponding to those elevations.

The recommended design discharges for the modelling, based on the analyses described above are provided in Table 2.

Table 2 – Recommended design discharges for Doris Creek bridge

Flood	Discharge
1:2 year flood peak	4.1 m^3/s
1:10 year flood peak	6.1 m^3/s
1:100 year flood peak	7.6 m^3/s
Design check	10 m^3/s

Table prepared by: BT *Table checked by:* NPS

Hydraulic Modeling

The channel geometry for the modelling was described previously. The HEC-RAS model was calibrated to an observed high water event on 28 June 2004, with a discharge of 1.363 m^3/s and a water surface elevation (adjusted to the bridge location) of 21.58 m. The calibration yielded a channel slope estimate of 0.004 m/m and a channel roughness estimate of n = 0.045. The cross-sectional data at the crossing were repeated at 20 m intervals, adjusting for channel slope, to model the creek channel upstream and downstream of the bridge.

The model was run for both a natural creek (no bridge) and with a bridge situation to estimate water elevations and velocities at the bridge site. To test the sensitivity of the model to channel roughness, the natural creek situation was also run with roughness values of 0.035 and 0.055. Results of the model runs are summarized in Tables 3 to 5.

Table 3 – Variability of water surface elevation and velocity with channel roughness (1:100 year design discharge)

Channel roughness	Water surface elevation at bridge crossing	Mean velocity (main channel)
n = 0.035	21.98 m	1.12 m/s
n = 0.045	22.07 m	0.96 m/s
n = 0.055	22.14 m	0.84 m/s

Table prepared by: BT *Table checked by:* NPS

Table 4 – Water surface elevations and velocities for the natural channel (Manning's roughness n = 0.045)

Design discharge	Water surface elevation at bridge crossing	Mean velocity (main channel)
1.36 m^3/s (calibration)	21.58 m	0.60 m/s
4.1 m^3/s (1:2-year flood)	21.87 m	0.78 m/s
6.1 m^3/s (1:10-year flood)	21.99 m	0.88 m/s
7.6 m^3/s (1:100-year flood)	22.07 m	0.96 m/s
10 m^3/s (design check)	22.17 m	1.06 m/s

Table prepared by: BT *Table checked by:* NPS

Table 5 – Water surface elevations and velocities with the proposed bridge (Manning's roughness n = 0.045)

Design discharge	Water surface elevation at bridge crossing	Mean velocity (main channel)
1.36 m^3/s (calibration)	21.58 m	0.60 m/s
4.1 m^3/s (1:2-year flood)	21.87 m	0.78 m/s
6.1 m^3/s (1:10-year flood)	21.99 m	0.88 m/s
7.6 m^3/s (1:100-year flood)	22.07 m	0.96 m/s
10 m^3/s (design check)	22.17 m	1.06 m/s

Table prepared by: BT *Table checked by:* NPS

The HEC-RAS modelling indicates that the channel will spill its banks at approximately the 1:10-year open water discharge, and at the 1:100-year open water discharge, the flow will be less than 0.1 m overbank. The water surface elevation at the design check discharge corresponds approximately to the observed historical high water mark at the bridge crossing.

The observed historical high water elevation of 22.2 m at the crossing compares well to the observed lake high water mark elevation of 22.27 m, as it is expected that there would be some drop in elevation from the lake water surface to the downstream channel. Thus, the calculated design water surface elevation and velocities presented in Table 5 are recommended for bridge design purposes. Corresponding water surface elevations are shown on Figure 2.

Given that the bridge will span the channel with several metres of setback from the top of bank to the toe of the headslope fill, and that mean channel velocities are expected to be on the order of only 1 m/s, no in-channel erosion protection is recommended for this bridge crossing.

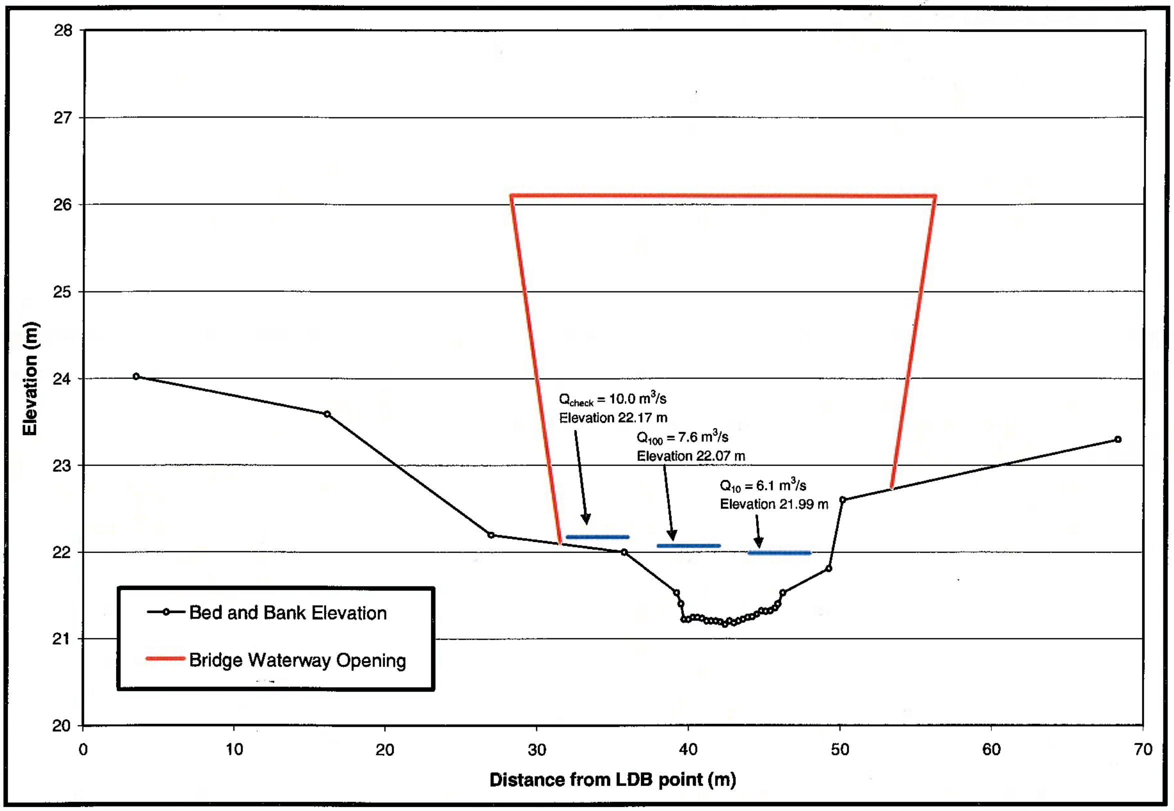

Figure 2 – Design water surface elevations at Doris Creek bridge

Closure

This hydrotechnical assessment is provided to SRK for use on the MHBL Doris North project infrastructure design and water license application to the Nunavut Water Board. Please contact the undersigned if you have any questions or concerns.

GOLDER ASSOCIATES LTD.

Nathan Schmidt

REGISTERED PROFESSIONAL ENGINEER
19 JAN 07
N.P. SCHMIDT
LICENSEE
N.W.T.

for Bernard Trevor, M.Eng., E.I.T.
Water Resources Engineer

Nathan Schmidt, Ph.D., P.Eng.
Associate, Senior Water Resources Engineer

References

AMEC. 2003. Meteorology and hydrology baseline report. Doris North Project, Revision 1, Supporting Document “D” to the Final Environmental Impact Statement. Prepared for Miramar Hope Bay Ltd. by AMEC Earth and Environmental, Burnaby, B.C., August 2003.

Environment Canada. 1994. Consolidated Frequency Analysis (CFA) Version 3.1. Software distributed by Greenland Engineering Group, Concord, Ontario.

Golder Associates Ltd. 2005. Doris North aquatic studies, 2004. Prepared for Miramar Hope Bay Ltd., North Vancouver, BC by Golder Associates Ltd., Edmonton AB. Golder Report 04-1373-009F (Rev. May 2005). 82 p + 3 app.

Golder Associates Ltd. 2006. Doris North aquatic studies, 2005. DRAFT report prepared for Miramar Hope Bay Ltd., North Vancouver, BC by Golder Associates Ltd., Edmonton AB. Golder Report 05-1373-014. 103 p + 3 app.

Golder Associates Ltd. 2007. Doris North aquatic studies, 2006. DRAFT report prepared for Miramar Hope Bay Ltd., North Vancouver, BC by Golder Associates Ltd., Edmonton AB. Golder Report 06-1373-026 in progress.

Rescan. 2002. 1993-2002 Data Compilation Report for Meteorology and Hydrology. Prepared for the Hope Bay Joint Venture by Rescan Environmental Services Ltd., May 2002.

APPENDIX I

Hydrological Analysis

Snowmelt Flood Estimates

Flood discharges were estimated based on the following method:

1) Observed concurrent flood peak discharges and snowpack snow water equivalents were compiled and used to derive a relationship.
2) Snow water equivalents for a long-term record were estimated based on regional data.
3) Flood peak discharges were calculated based on the long-term record of snow water equivalents.
4) A frequency analysis of the flood peak discharges was performed to estimate floods of specific return periods.

Available concurrent flood peak and snowpack data for the project area are available as shown in Table I-1 and Figure I-1.

Table I-1 – Concurrent flood peak and snowpack data at Doris North

Year	Doris Creek Flood Peak	Doris Creek Watershed Snow Water Equivalent
1997[a]	5.46 m^3/s	104 mm
1998[a]	5.60 m^3/s	90 mm
2004[b]	1.36 m^3/s	56 mm
2005[b]	1.98 m^3/s	66 mm
2006[b]	2.72 m^3/s	79 mm

Table prepared by: NPS *Table checked by:* AJS

(a) From Rescan (2002)

(b) From Golder (2005, 2006, 2007)

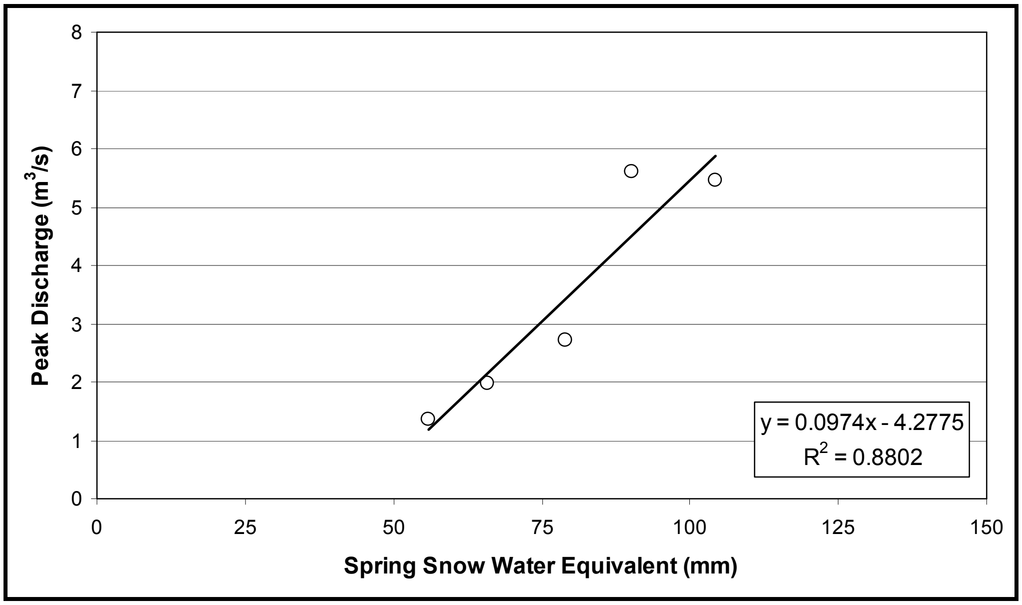

Figure I-1 – Concurrent flood peak and snowpack data at Doris North

Snow water equivalents were estimated based on the derived snowfall record presented by AMEC (2003). Each annual snowfall depth was corrected for undercatch by multiplying by a factor of 1.71, as estimated by AMEC (2003), and further corrected for sublimation by multiplying by a factor of 0.70.

Table I-2 – Long-term estimates of annual snow water equivalents and flood peak discharges

Year	Snowfall (cm)	Undercatch Corrected Snowfall (mm)	Sublimation Corrected Snow Water Equivalent (mm)	Estimated Annual Flood Peak Discharge (m^3/s)
1959	86.5	147.9	103.5	5.81
1960	78.2	133.7	93.6	4.84
1961	61.2	104.7	73.3	2.86
1962	59.1	101.1	70.7	2.61
1963	73.7	126.0	88.2	4.32
1964	68.2	116.6	81.6	3.67
1965	51.7	88.4	61.9	1.75
1966	28.3	48.4	33.9	(a)
1967	68.7	117.5	82.2	3.73
1968	90.2	154.2	108.0	6.24
1969	48.6	83.1	58.2	1.39
1970	58.4	99.9	69.9	2.53
1971	85.5	146.2	102.3	5.69
1972	78	133.4	93.4	4.82
1973	55.2	94.4	66.1	2.16
1974	81.1	138.7	97.1	5.18
1975	62.3	106.5	74.6	2.99
1976	64.9	111.0	77.7	3.29
1977	74.5	127.4	89.2	4.41
1978	78.5	134.2	94.0	4.87
1979	59.6	101.9	71.3	2.67
1980	53.3	91.1	63.8	1.94
1981	56.6	96.8	67.8	2.32
1982	63.8	109.1	76.4	3.16
1983	83	141.9	99.4	5.40
1984	62.5	106.9	74.8	3.01
1985	76.7	131.2	91.8	4.66
1986	81.9	140.0	98.0	5.27
1987	79.7	136.3	95.4	5.01
1988	44.4	75.9	53.1	0.90
1989	70.9	121.2	84.9	3.99
1990	59.7	102.1	71.5	2.68
1991	93.7	160.2	112.2	6.65
1992	97.4	166.6	116.6	7.08
1993	74.7	127.7	89.4	4.43
1994	66.2	113.2	79.2	3.44
1995	86.8	148.4	103.9	5.84
1996	68	116.3	81.4	3.65
1997	76.8	131.3	91.9	4.68
1998	82.7	141.4	99.0	5.36
1999	86.4	147.7	103.4	5.80
2000	76.3	130.5	91.3	4.62
2001	90.8	155.3	108.7	6.31
2002	71	121.4	85.0	4.00

Table prepared by: NPS *Table checked by:* AJS

(a) Low snowfall predicts no snowmelt flood.

A frequency analysis of the flood peak estimates contained in Table I-2 was performed using the Generalized Extreme Value (GEV) and Log Pearson Type 3 (LP3) distributions (Environment Canada 1994). The results are shown in Table I-3. GEV estimates, being larger than the LP3 estimates, are the more conservative of the results.

Table I-3 – Flood frequency analysis results

Return Period (years)	GEV Flood Estimate (m^3/s)	LP3 Flood Estimate (m^3/s)
2	4.09	4.16
5	5.43	5.43
10	6.12	6.01
20	6.67	6.43
50	7.24	6.83
100	7.58	7.04
200	7.87	7.21

Table prepared by: NPS *Table checked by:* AJS

Lake High Water Mark Estimates

During field work in August 2006, high water marks were observed on a rock face located approximately 100 m south of the Doris Lake hydrometric station. A photograph taken that day is shown in Figure I-2, and elevation and discharge estimates, tied in on the basis of lake water surface elevation data and the lake outlet rating curve, are provided in Table I-4.

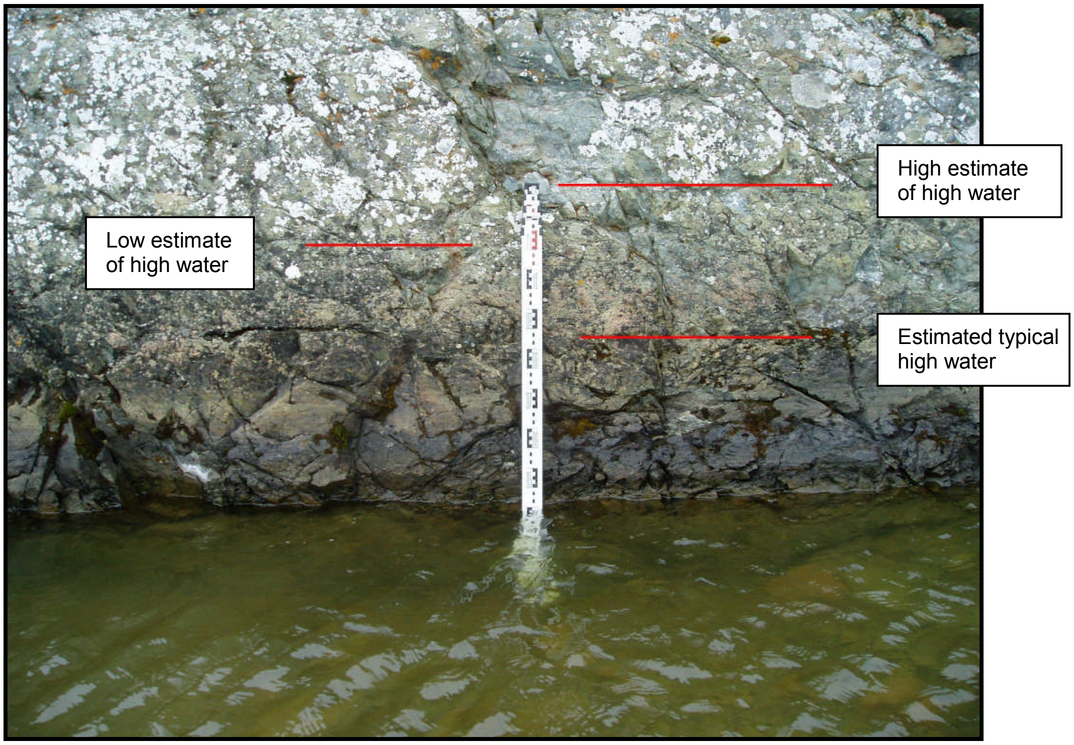

Figure I-2 – High water marks at Doris Lake, August 2006

Table I-4 – High water mark elevations at Doris Lake, August 2006

Mark	Rod Reading (m)	Elevation (m)	Lake Stage (m)	Estimated Discharge (m^3/s)
Doris Lake WL 11 August 2006	0.375	21.448	0.448	0.243
Typical High Water (no lichen observed below this level)	0.830	21.903	0.903	2.084
Low Estimate of High Water	1.060	22.133	1.133	4.180
High Estimate of High Water	1.200	22.273	1.273	5.976

Table prepared by: NPS *Table checked by:* AJS

Estimates of lake discharge based on observed lake high water marks indicate a high water discharge of approximately 6 m^3/s. However, these are based on the absence of lichen from lakeside rocks and may not be indicative of an event with a return period as large as 100 years. Furthermore, the lake outlet rating curve is based on monitoring data for the period 2004 to 2006, and is only confirmed for discharges below 2.7 m^3/s. Thus, uncertainty exists with regards to the accuracy of these estimates.

Conclusions

As previously stated, hydrometric data for the local area are sparse, and high flow data are even more so. However, two methods yield design peak discharge estimates that are broadly consistent (7.6 m^3/s using the snowpack method and 6.0 m^3/s using the high water mark method) and design high water at the bridge must by definition be lower than that at the lake.

It is recommended that the bridge design use the calculated 100-year discharge of 7.6 m^3/s as a design discharge with an additional check discharge of 10 m^3/s.

Appendix H

Technical memrandum on liner and geotextile requirements for fuel tank farm

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Technical Memorandum

To:	Jim Currie	**Date:**	March 10, 2007
cc:	Project File	**From:**	Maritz Rykaart
Subject:	Design assumptions for selecting HDPE liner and geotextile for use in the Doris North Project	**Project #:**	1CM014.008

1 Introduction

This memo documents the reasoning behind selection of the following geosynthetic products for use in the Doris North Project:

- 1.4 mm (57 mil) textured HDPE liner for containment in the fuel transfer station, fuel tank farm and the pollution control pond.
- 385 g/m^2 (12 oz) non-woven geotextile for protection of the HDPE liner.

These geosynthetics will be used in the following areas:

- Fuel transfer station
- Fuel tank farm
- Pollution control pond
- Landfarm
- Emergency Dump Catch Basins

2 Liner Rationale

The purpose of the liner to be used on site, for all intended applications is full containment of various streams of potential deleterious substances. Since there are no suitable natural materials with which to construct natural containment structures, the use of geosynthetics is considered appropriate. Some of the important design criteria considered in the choice of liner are as follows:

- The material must be compatible for use in the arctic.
- The material must be compatible against all known chemical compounds to which it may be subjected, including hydrocarbons and various streams of water containing varying degrees of acidity and dissolved metals.
- It must be possible to safely and effectively install the liner at extremely low temperatures.
- The material must be sufficiently robust that it can withstand the rigours of remote artic construction, under less than ideal conditions.
- The material must have a proven track record for use in similar applications, under similar conditions.
- If at all possible, a single material must be specified for all facilities, such that more effective product use can be insured.
- There must be a qualitative and quantitative standardized procedure to measure the installation quality of the product.

- Although the product is only expected to be used for a short design life, the product must be guaranteed to last the duration of the mine life.
- If damaged, either during construction, or later during operation, the product must lend itself to being completely and effectively repaired locally on site, without having to completely replace the liner.
- Installation of the liner must be sufficiently flexible, such that small design changes or deviations from design grades can easily be accommodated.
- The product must be readily available.

Given all of the above mentioned design criteria, HDPE liner was selected as the preferred product. A 1.4 mm (57 mil) thick product was selected, since it is sufficiently thick to withstand the rigours of construction, but still allow relative ease of installation. To facilitate safe and efficient installation, textured liner has been selected, such that the textured surface can be used as a traffic surface to prevent slips and falls during winter construction.

This type of liner is commonly used for this kind of application and in this environment, and therefore there is ample precedent.

3 Geotextile Rationale

The primary purpose of the geotextile is to act as a liner protection layer. Again, since there is no suitable fine sand to use as a liner protection layer, the use of a geotextile is considered appropriate, and in any event is considered standard practice.

Important design criteria in the use of the type of geotextile are as follows:

- The material must be compatible for use in the arctic.
- The material must be compatible against all known chemical compounds to which it may be subjected, including hydrocarbons and various streams of water containing varying degrees of acidity and dissolved metals.
- The material must be sufficiently robust that it can withstand the rigours of remote artic construction, under less than ideal conditions.
- The material must have a proven track record for use in similar applications, under similar conditions.
- If at all possible, a single material must be specified for all facilities, such that more effective product use can be insured. The product must however also be economical enough that if conditions require double layers can be placed.
- Although the product is only expected to be used for a short design life, the product must be guaranteed to last the duration of the mine life.
- The product must be readily available.

Given the material types against which the geotextile must provide liner protection, a relatively heavy geotextile such as a 16-oz geotextile, would be preferred. However, taking due care in preparation of the liner bedding and cover material layers would allow for a lesser demand on the geotextile, and therefore it was decided to specify a lower grade geotextile, i.e. the 12-oz fabric. This fabric is economical enough that double layering would be possible when conditions require it. This is a call the Engineer will make on site in accordance with the QA/QC procedures.

4 Installation QA/QC

Supporting Documents SD-01, SD-02, SD-03 and SD-04 describes complete details of where and how these geosynthetic products are to be used. Under all circumstances installation of HDPE liner can only be carried out by a Specialist Contractor qualified to install liner in accordance with the stipulated Project Specifications. All liner installation and testing procedures are stipulated on the Engineering Drawings (SD-04) as well as in the Project Specifications (SD-03), and will be enforced on site by the Engineer’s Representative for the Engineer of Record.